DESIGN AND COMPARITIVE ANALYSIS OF REINFORCED CONCRETE STRUCTURE AND STEEL STRUCTURE

PROJECT REPORT

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BONAFIDE CERTIFICATE

Certified that this project report **"DESIGN AND COMPARITIVE ANALYSIS OF REINFORCED CONCRETE STRUCTURE AND STEEL STRUCTURE"** is the bonafide work of ………………………………………………….. who carried out the project work under my supervision.

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Wishes.

TABLE OF CONTENTS

A.ABSTRACT

The main aim of this project is to gain knowledge on the analysis and design of a reinforced concrete structure and a steel structure and to analyze the pros and cons for both the material in construction. A residential building of **Ground+1** floor is chosen for this project and the complete planning

The proposed residential building is to be implemented at Sivasakthi Nagar, in Saravanampatti, Coimbatore. The proposed residential building is a framed structure.

The load calculations have been done as per IS 875-1987(PART I & II) and the manual design is done as per IS 456-2000 (concrete) and IS 800-1987 (steel).These calculations have been supplemented with the output from STADD. Pro software. Detailed drawing showing the reinforcement details for different members are presented using the Auto CAD software.

Which is more sustainable - concrete or steel-framed buildings? This is a question people have asked themselves time and time again and, as a result, many comparisons have been made. Owners and project teams must work together to evaluate every aspect of a proposed project to deliver "the most bang for the buck." The comparison has been carried out using the following parameters

- Property wise comparison
- Cost wise comparison
- Environmental consideration

B.LIST OF TABLE

Page No

C.LIST OF FIGURES **Page No**

D.LIST OF SYMBOLS

The following symbols carrying the meanings noted against them are used in this volume.

A	= Area
A_{st}	= Area of the steel reinforcement
BM	= Bending Moment
B	= Breadth of the beam, slab and shorter span
D	= Over all depth of beam or slab
b	= Breath of column
d	= Effective length of the beam or slab
f_y	= Characteristic strength of steel
f_{ck}	= Characteristic compressive strength of concrete
l	= Length of the beam
l_x	= Length of shorter span
l_y	= Length of longer span
l_{ex}	= Effective length of slab along shorter span
l_{ey}	= Effective length of slab along longer span
M_x, M_y	= Moments on the strip of unit width spanning l_y and l_x
M_{ux}, M_{uy}	= Moments about x and y axes due to design loads
M_{ux1}, M_{uy1}	= Maximum uniaxial moments capacity for an axial load of Pu
MOR	= Moment of resistance
P_u	= Axial load on a compression member

S_v	= Spacing of stirrups
V	= Shear force
V_s	= Shear force (design)
W	= Total load
α_x	= Bending moment co-efficient along shorter span
α_y	= Bending moment co-efficient along longer span
τ_v	= Shear stress in concrete (permissible)
τ_c	= Shear stress in concrete (maximum)
ø	= Diameter of bars
N	= Newton
mm	= Millimeter
M	= Meter
c/c	= Center to center
Fe415	= High yield strength deformed bars
M20	= Grade of concrete
DL	= Dead load
LL	= Live load
IL	= Imposed load

1. INTRODUCTION

This project seeks to compare a steel-framed residential building using composite floor construction – concrete floors supported on steel beams – with, concrete frame with in situ concrete floors. There are many major materials used for construction of residential buildings. This project aims to compare the feasibility of usage of steel for residential buildings over the conventional reinforced concrete.

Where do we find steel in modern residential construction?

- Concrete foundations and raft slabs in most houses contain a few tones of rebar or reinforcing steel, nicely hidden away but never the less doing an excellent job.
- Houses which are raised off the ground use steel columns, steel floor bearers and in many cases steel floor joists. (As in steel purlins).
- Steel residential wall framing is extremely strong, lightweight and very cost effective. The system of bracing it is simple and strong. It can take all the traditional siding, cladding or sheeting materials.
- The wall framing itself can incorporate steel RHS (rectangular hollow sections) around openings to provide extra strength, which carry the roof loads to the foundations.
- Whatever the type of wall construction, the roof structure almost always has some steel in it, be it a couple of steel beams over patios, bolts and angle brackets to conventional timber or truss framing.
- More often we are seeing complete steel truss layouts, with steel battens and steel roof sheeting.

This project replaces reinforced concrete with steel only for the load bearing structures and is intended to design a composite structure with R.C roof, steel beams, columns and column bases.

Use of steel as a construction material has seen phenomenal growth in the last few years. Although steel buildings have been used in the commercial and industrial sectors for a long time, they are increasingly gracing the skylines of countries all over the world. Steel buildings were once considered to belong in the category of industrial and commercial buildings. Their strength and advantages, however, have now been seen to offer great advantages for residential applications too. With more and more people looking for affordable housing that can be provided quickly, residential steel buildings are gaining an increasingly popular reputation in the real estate world. The advantages of residential steel buildings, also known as residential modular buildings, include the fact that most of the preliminary construction work is carried out at a remote site, usually a factory, where the various part of the building are prefabricated. So which option is best, steel or concrete? Depending on the specifics of a particular project, both! Owners must consider several factors before this question can be properly answered.

1.1 LOAD CALCULATIONS

Load Data

Floor to floor height	= 3.0 m
Depth of foundation	= 1.8m
Safe bearing capacity of the soil	= 230 KN/m^2
Concrete grade	= M20
Steel grade	= Fe 415
Unit weight of concrete	= 25 KN/m^2
Unit weight of brick masonry	= 20 KN/m^2

Design: Limit State Method

Code: IS 456: 2000

Assumed Imposed Loads As per IS 875: 1987 (Part II)

ROOF:

Roof finish	= 0.75 KN/m^2
Live load	= 3KN/m^2
Total load	= 3.75 KN/m^2

FLOOR:

Floor finish	= 0.75 KN/m^2
Live load	= 3 KN/m^2
Total load	= 3.75 KN/m^2

Dead Load Calculations

SLAB:

Slab Thickness = 125mm

Self Weight of Slab = 3.125 KN/m^2

WALL:

Main Wall Thickness = 230mm

Partition Wall Thickness = 110mm

Self Weight of main Wall = 13.8 KN/m

Self Weight of partition wall = 6.6 KN/m

BEAM:

Main beam = 230mm x 440mm

Self Weight of Main Beam = 2.53 KN/m

COLUMN:

Column Size = 230mm x 230mm

Self weight Of Column = 4KN/m

Load combination

Analysis has been done for the building frame using STAAD Pro software, to find out the bending moment shear force and deflection produced on the members due to applied loads.
The load combination used in the analysis is:

1. Dead load + Live load

The results obtained from the STAAD output have been for the design of slabs, columns, beams, and footings of the building.

1.2 STAAD PRO RESULTS – R.C STRUCTURE

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B E A M N O.   97 D E S I G N R E S U L T S

M20   Fe415 (Main)   Fe415 (Sec.)

LENGTH: 3300.0 mm   SIZE: 230.0 mm X 300.0 mm   COVER: 25.0 mm

SUMMARY OF REINF. AREA (Sq.mm)

SECTION	0.0 mm	825.0 mm	1650.0 mm	2475.0 mm	3300.0 mm
TOP REINF.	341.36 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	371.16 (Sq. mm)
BOTTOM REINF.	0.00 (Sq. mm)	126.72 (Sq. mm)	189.64 (Sq. mm)	126.72 (Sq. mm)	0.00 (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

SECTION	0.0 mm	825.0 mm	1650.0 mm	2475.0 mm	3300.0 mm
TOP REINF.	5-10í 1 layer(s)	2-10í 1 layer(s)	2-10í 1 layer(s)	2-10í 1 layer(s)	5-10í 1 layer(s)
BOTTOM REINF.	2-12í 1 layer(s)	2-12í 1 layer(s)	2-12í 1 layer(s)	2-12í 1 layer(s)	2-12í 1 layer(s)
SHEAR REINF.	2 legged 8í @ 100 mm c/c	2 legged 8í @ 100 mm c/c	2 legged 8í @ 100 mm c/c	2 legged 8í @ 100 mm c/c	2 legged 8í @ 100 mm c/c

SHEAR DESIGN RESULTS AT DISTANCE d (EFFECTIVE DEPTH) FROM FACE OF THE SUPPORT

SHEAR DESIGN RESULTS AT 380.0 mm AWAY FROM START SUPPORT
VY = 43.45 MX = -0.17 LD= 6
Provide 2 Legged 8í @ 100 mm c/c

SHEAR DESIGN RESULTS AT 380.0 mm AWAY FROM END SUPPORT
VY = -46.17 MX = -0.17 LD= 6
Provide 2 Legged 8í @ 100 mm c/c

===

B E A M N O. 102 D E S I G N R E S U L T S

M20 Fe415 (Main) Fe415 (Sec.)

LENGTH: 3000.0 mm SIZE: 230.0 mm X 300.0 mm COVER: 25.0 mm

SUMMARY OF REINF. AREA (Sq.mm)

SECTION	0.0 mm	750.0 mm	1500.0 mm	2250.0 mm	3000.0 mm
TOP REINF.	343.65 (Sq. mm)	127.19 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	236.59 (Sq. mm)
BOTTOM REINF.	0.00 (Sq. mm)	127.19 (Sq. mm)	148.73 (Sq. mm)	127.19 (Sq. mm)	0.00 (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

SECTION	0.0 mm	750.0 mm	1500.0 mm	2250.0 mm	3000.0 mm
TOP	5-10í	2-10í	2-10í	2-10í	4-10í

REINF. 1 layer(s) 1 layer(s) 1 layer(s) 1 layer(s) 1 layer(s)

BOTTOM 2-10í 2-10í 2-10í 2-10í 2-10í
REINF. 1 layer(s) 1 layer(s) 1 layer(s) 1 layer(s) 1 layer(s)

SHEAR 2 legged 8í 2 legged 8í 2 legged 8í 2 legged 8í 2 legged 8í
REINF. @ 100 mm c/c @ 100 mm c/c @ 100 mm c/c @ 100 mm c/c @ 100 mm c/c

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SHEAR DESIGN RESULTS AT DISTANCE d (EFFECTIVE DEPTH) FROM FACE OF THE SUPPORT

SHEAR DESIGN RESULTS AT 380.0 mm AWAY FROM START SUPPORT
VY = 42.49 MX = -0.23 LD= 6
Provide 2 Legged 8í @ 100 mm c/c

SHEAR DESIGN RESULTS AT 380.0 mm AWAY FROM END SUPPORT
VY = -35.24 MX = -0.23 LD= 6
Provide 2 Legged 8í @ 100 mm c/c

===

B E A M N O. 107 D E S I G N R E S U L T S

M20 Fe415 (Main) Fe415 (Sec.)

LENGTH: 2700.0 mm SIZE: 230.0 mm X 300.0 mm COVER: 25.0 mm

SUMMARY OF REINF. AREA (Sq.mm)

SECTION	0.0 mm	675.0 mm	1350.0 mm	2025.0 mm	2700.0 mm
TOP REINF.	217.12 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	188.38 (Sq. mm)
BOTTOM REINF.	0.00 (Sq. mm)	127.19 (Sq. mm)	127.19 (Sq. mm)	127.19 (Sq. mm)	127.19 (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

SECTION	0.0 mm	675.0 mm	1350.0 mm	2025.0 mm	2700.0 mm
TOP REINF.	2-12í 1 layer(s)	2-12í 1 layer(s)	2-12í 1 layer(s)	2-12í 1 layer(s)	2-12í 1 layer(s)
BOTTOM REINF.	2-10í 1 layer(s)	2-10í 1 layer(s)	2-10í 1 layer(s)	2-10í 1 layer(s)	2-10í 1 layer(s)
SHEAR REINF.	2 legged 8í @ 100 mm c/c	2 legged 8í @ 100 mm c/c	2 legged 8í @ 100 mm c/c	2 legged 8í @ 100 mm c/c	2 legged 8í @ 100 mm c/c

SHEAR DESIGN RESULTS AT DISTANCE d (EFFECTIVE DEPTH) FROM FACE OF THE SUPPORT

SHEAR DESIGN RESULTS AT 380.0 mm AWAY FROM START SUPPORT

VY = 32.64 MX = 0.10 LD= 6

Provide 2 Legged 8í @ 100 mm c/c

SHEAR DESIGN RESULTS AT 380.0 mm AWAY FROM END SUPPORT

VY = -30.61 MX = 0.10 LD= 6

Provide 2 Legged 8í @ 100 mm c/c

===

COLUMN NO. 18 DESIGN RESULTS

M20 Fe415 (Main) Fe415 (Sec.)

LENGTH: 1800.0 mm CROSS SECTION: 230.0 mm X 230.0 mm COVER: 40.0 mm

** GUIDING LOAD CASE: 6 END JOINT: 18 SHORT COLUMN

REQD. STEEL AREA : 673.14 Sq.mm.
REQD. CONCRETE AREA: 52226.87 Sq.mm.
MAIN REINFORCEMENT : Provide 4 - 16 dia. (1.52%, 804.25 Sq.mm.)
(Equally distributed)
TIE REINFORCEMENT : Provide 8 mm dia. rectangular ties @ 230 mm c/c

SECTION CAPACITY BASED ON REINFORCEMENT REQUIRED (KNS-MET)

Puz : 914.58 Muz1 : 19.47 Muy1 : 19.47

INTERACTION RATIO: 0.97 (as per Cl. 39.6, IS456:2000)

SECTION CAPACITY BASED ON REINFORCEMENT PROVIDED (KNS-MET)

WORST LOAD CASE: 6
END JOINT: 18 Puz : 953.61 Muz : 23.05 Muy : 23.05 IR: 0.73

1.3 FRAME ANALYSIS

A building frame is a complicated statically indeterminate structure. The analysis by the moment distribution method is very lengthy and difficult. There are lots of methods for analyzing a frame. But the method we use is Substitute frame method which is the easy method.

In this method only a part of frame is considered for analysis. The considered part is called Substitute frame. The moments for each floor are separately computed. It will be assumed that the moments transferred from one floor to another are small floor. Each floor will be taken as connected to columns above and below with their far ends fixed. The frame taken this way is analyzed for the moments and shears in the beams and columns.

LOAD CALCULATION:

Span AB:

Influence area	$= ((4.12 + 2.505)/2) * 1.615 + 4.12 * 1.615$
	$= 12m^2$
Self weight of slab	= 12 * 0.15 * 25000
	= 45000 N/m
Self weight of beam	= 4.12 * 0.23 * 0.3 * 25000
	= 7107N/m
Self weight of wall	= 4.12 * 0.23 * 3 * 25000
	= 56856 N/m
Floor Finish	= 12 * 750
	=9000 N/m
Dead load	= 28631.8 N/m
Live load	= 8737.86N/m

DESIGN LOAD:

Dead load	= 42948 N/m
Live load	= 13136.79 N/m

Span BC:

Influence area	= (1 / 2) * 1.615 * 3.23 + 3.23* 1.465
	= 7.34 m^2
Self weight of slab	= 7.34 * 0.15 * 25000
	= 27525 KN/m
Self weight of beam	= 3.23 * 0.23 *0.3* 25000
	= 5571.5 KN/m
Self weight of wall	=3.23* 0.23 * 3* 20000
	= 44574 KN/m
Floor Finish	= 7.34*750
	=5505 N/m

DESIGN LOAD:

Dead load	= 25751 N/m
Live load	= 6817.33 N/m

Span CD:

Influence area	= ((2.93+1.315)/2)*1.615+2.93*1.456*1/2
	= 5.574 m^2
Self weight of slab	= 5.574 * 0.15 * 25000
	= 20902.5 N/m
Self weight of beam	= 2.93 * 0.23 *0.3* 25000
	= 5054.25 N/m

Self weight of wall =2.93* 0.23 * 3* 20000

= 40434KN/m

Floor Finish = 2.93*750

=4180.5 N/m

DESIGN LOAD:

Dead load = 24085.7 N/m

Live load = 5707.17 N/m

CALCULATION OF FIXED END MOMENT: (1.3.a)

Span	AB	BC	CD
Length	4.12	3.23	2. 93
Fixed end moment due to DL	60751.37	33582.2	25846.72
Fixed end moment due to DL+LL	79333.84	42472.7	31971.16

CALCULATION OF DISTRIBUTION FACTOR: (1.3.b)

JOINT	RS	TOTAL RS	DF
A	AB=0.243I AI=0.333I AE=0.333I	0.909I	0.297 0.366 0.366
B	BA=0.243I BJ=0.333I BF=0.333I BC=0.309I	1.218I	0.200 0.273 0.273 0.254
C	CB=0.309I CG=0.333I CK=0.333I CD=0.341I	1.31I	0.236 0.252 0.252 0.260
D	DC=0.341I DL=0.3333I DH=0.333I	1.00I	0.341 0.323 0.323

JOINT - A: (1.3.c)

Joint	A	B		C		D
Member	AB	BA	BC	CB	CD	DC
DF	0.267	0.2	0.254	0.236	0.26	0.341
DL FEM			-33582.8			
TL FEM	-79333.84	+79333.84				
DIS&COM	-4575.164					
NET	-83909.004					
Distribution	+22403.7					
Net Moment	-61505.30					

JOINT - B: (1.3.d)

Joint	A	B		C		D
Member	AB	BA	BC	CB	CD	DC
DF	0.267	0.2	0.254	0.236	0.26	0.341
DL FEM					-25846.72	25846.72
TL FEM	-79333.84	+79333.84	-42472.76	+42472.76		
DIS&COM		10591.07	-2111.46			
		89924.91	-44584.22			
Distribution		-9068.142	11516.54			
Net Moment		80856.77	56100.76			

JOINT - C: (1.3.e)

Member	AB	BA	BC	CB	CD	DC
DF	0.267	0.2	0.254	0.236	0.26	0.341
DL FEM	-60751.37	60751.37			-25846.72	25846.72
TL FEM			-42472.76	+42472.76	-31971.16	31971.16
DIS&COM				2156.911	-4156.25	
				44629.37	-36127.41	
Distribution				-10532.53	-9393.02	
Net Moment				34096.82	-26734.39	

JOINT - D: (1.3.f)

Member	AB	BA	BC	CB	CD	DC
DF	0.267	0.2	0.254	0.236	0.26	0.341
DL FEM	-60751.37	60751.37	-33582.2	33582.2		
TL FEM					-31971.16	31971.16
DIS&COM						273.87
						32245.04
Distribution						-10963.31
Net Moment						21281.13

SPAN - AB: (1.3.g)

Joint	A	B		C		D
Member	AB	BA	BC	CB	CD	DC
DF	0.267	0.2	0.254	0.236	0.26	0.341
DL FEM			-33582.2	33582.2		
TL FEM	-79333.84	+79333.84			-31971.16	31971.16
DIS	21182.135	-9150.328				
COM	-4575.164	10591.07				
Distribution	1221.57	-2118.214				
Net Moment	-61505.299	78656.37				

Net Bending Moment = $Wl^2/8$

$= (1.5*79333.84)-((615805.299+78656.37)/2)$

$= 48919.92$ N-m

SPAN - BC (1.3.h)

Joint	A	B		C		D
Member	AB	BA	BC	CB	CD	DC
DF	0.267	0.2	0.254	0.236	0.26	0.341
DL FEM	-60751.37	60751.37			-25846.72	25846.72
TL FEM			-42472.76	+42472.76		
DIS	16220.6	-3655.76	-3655.76	-2478.37	-2730.30	-10902.31
COM		8310.3	-1239.19	-1827.88	-5451.16	
Distribution			1414.22	-1728.47		
Net Moment			-46303.55	36438.04		

Net Bending Moment = $Wl^2/8$

= (1.5*42472.76)-((36438.04+46303.55)/2)

= 22338.34 N-m

SPAN - CD: (1.3.i)

Joint	A	B		C		D
Member	AB	BA	BC	CB	CD	DC
DF	0.267	0.2	0.254	0.236	0.26	0.341
DL FEM			-33582.2	33582.2		
TL FEM	-79333.84	+79333.84			-31971.16	31971.16
DIS	21182.14	-9150.328	-17385.6	10797.3	-418.87	-10902.23
COM				-8698.8	-5451.11	-209.44
Distribution					3677.42	-71.419
Net Moment					-34163.73	20788.27

Net Bending Moment = $Wl^2/8$

= (1.5*31971.16)-((34163.73+20788.27)/2)

= 20480.74 N-m

MOMENT IN COLUMN DUE TO TL FEM: (1.3.j)

JOINT	A	B		C		D
MEMBER	AB	BA	BC	CB	CD	
DF	0.267	0.2	0.254	0.236	0.26	0.341
TL FEM	-79333.84	+79333.84	-42472.76	+42472.76	-31971.16	31971.16
DIS & COM	-3686.108	10591.07	4681.36	1239.19	5435.10	1365.2
ADD	-83019.95	89924.91	-37791.4	43711.95	-26536.06	33336.36
TOP COL	30385.30		10317.05	-10317.05		10767.64
BOT COL	30385.30		10317.05	-10317.05		10767.64

MOMENT IN COLUMN DUE TO DL FEM: (1.3.k)

JOINT	A	B		C		D
MEMBER	AB	BA	BC	CB	CD	
DF	0.267	0.2	0.254	0.236	0.26	0.341
DL FEM	-60751.37	60751.37	-33582.2	33582.2	-25846.72	25846.72
DIS & COM	-2717.07	8110.31	-912.79	-3450.61	4406.84	-1005.61
ADD	-63468.43	68862.68	-34494.99	30131.59	-21439.85	-24841.69
TOP COL	16946.07		-75931.6	75931.6		80236.9
BOT COL	16946.07		-75931.6	75931.6		80236.9

CONCRETE DESIGN

1.4 DESIGN OF SLAB

METHODS OF DESIGN:

Structural member are designed by using the following methods,

1. Working stress method – load is constant & stress is reduced.
2. Limit state method – load is increased & stress is reduced.
3. Ultimate strength method – load is increased & stress is reduced.

Working stress method and limit state method is commonly used. In this project, the design is done using limit state method. The rules given in IS 456-2000 for plain and reinforced concrete has been adopted.

SLAB:

The slabs are built monolithically and support on either side by beams. There are two forms of slab. They are simply supported slab & Cantilever slab. The live load for typical floor slab is taken as 3kN/m2 and for roof slab, live load is taken as 1.5 kN/m2 as per National building code 1983. The characteristic strength of steel used is Fe-415 and characteristic strength of concrete used in M20.

The reinforced concrete slab supported on two parallel long edges only and free on two parallel short edges& aspect ratio is greater than 2, the slab is called one way slab. The reinforced concrete slab supported on all four edges and aspect ratio is less than 2, the slab is called two way slab.

Loads considered for design is:

a) Dead load

1. Self weight of structure

2. weight of finishes

b) live load

Slabs are plane structural members whose thickness is quite small as compared to its other dimensions. Slabs are most frequently used as roof coverings and floors in various shapes such as square, rectangular, circular and triangular in buildings, tank.

Some simple forms of slabs are

Cantilever slab

Simply supported slab

DESIGN PROCEDURE FOR TWO WAY SLAB:

Effective span of slab is taken least of following

a) centre to centre of support

b) clear span + effective depth

The bending moment per unit width of slab are given by following equation as per IS 456 2000 Appendix C.

$M_x = \alpha_x * w * l_x^2$

$M_y = \alpha_y * w * l_x^2$

α_x & α_y = Coefficient of bending moment

M_x & M_y = Moments on strips of unit width spanning lx and ly respectively

l_x = Length of the shorter span

l_y = Length of the long span

Slabs are considered as divided in each direction into middle strips and edge strips, the middle strip being there quarters of the width and each edge strip of the width, the reinforcement in middle strip and edge strip are calculated using SP-16 for moment of Resistance of slab.

SLAB : **HALL (GF&FF)**

TYPE : Two way slab, two short edges discontinuous

SIZE : Rectangular slab of 9.69m*6.23m

ASPECT RATIO : $l_x / l_y = 1.55<2$

ASSUME:

D=150mm & d=125mm & CLEAR COVER=25mm

LOAD:

Assume Live load	$=3kN/m^2$
Finishes	$=0.75kN/m^2$
Self weight of slab	$=0.15*1*1*25=3.75\ kN/m^2$
Partition wall	$=0.75kN/m^2$
Total load	$=8.25kN/m^2$
Factored load	=12.375 kN/m

BENDING MOMENT COEFFICIENT:

+ve moment

$\alpha_x = 0.0458$

$\alpha_y = 0.035$

-ve moment

$\alpha_x = 0.0458$

$\alpha_y = 0.035$

BENDING MOMENT:

$- M_x = 29.30$

$- M_y = \text{nil}$

$M_x = 21.99$

$M_y = 16.89$

CHECK FOR DETH:

M=Qbd2

For M-20 & Fe-415, Q=2.76

$d=(M/Qb)^{1/2}=103mm<125mm$

Therefore D=150mm and d=125mm

MAIN STEEL:

Assume 10mm φ bars

Short –ve span

Provide 10 mm dia at 100 mm c-c.

Long –ve span

nil

Long +ve span

Provide 10 mm dia at 195 mm c-c

Short +ve span

Provide 10 mm dia at 140 mm c-c

DISTRIBUTION STEEL:

$A_s = (0.12/100)*1000*150 = 180\ mm^2$

Provide 8mm @ 275mm c/c

TORSION REINFORCEMENT:

Torsion reinforcement at corners contained by two discontinuous edges

= ¾ of A_{st} max.

= $555mm^2$

Provide 8mm @ 100mm c/c for a length equal to 1.938m*1.938m.

SLAB : Dining hall.

TYPE : One way slab

SIZE : Rectangular slab of 7.12m *2.93m

ASPECT RATIO : $l_y / l_x = 2.43>2$

ASSUME:

D=150mm & d=125mm

CLEAR COVER=25mm

LOAD:

Assume Live load	$=3kN/m^2$
Finishes	$=0.75kN/m^2$
Self weight of slab	=0.15*1*1*25=3.75
Partition wall	$=0.75kN/m^2$
Total load	$=8.25kN/m^2$
Factored load	=12.375 KN/m

$B.M = (w_{lx}^2/8)$

$B.M = 13.28$ KN-m

As per design aids

$(Mu,lim/bd^2) = 2.76$

d=69.37 mm

Provide d=125 mm

d=150 mm

As per annex G of IS 456

$M_u = 0.87\, fy\, A_{st}\, d\, (1 - 1.66*10^{-4} A_{st})$

$13.28*10^6 = 45131.25\, A_{st}(1-1.66*10\text{-}4\, A_{st})$

$A_{st} = 294\ mm^2$

Provide 8 mm dia

Spacing=150 mm

Provide 8 mm dia at a spacing 0f 150 mm c-c

DISTRIBUTION STEEL:

$A_s = (0.12/100)*1000*150 = 180\ mm^2$

Provide 8mm @ 275mm c/c

TORSION REINFORCEMENT:

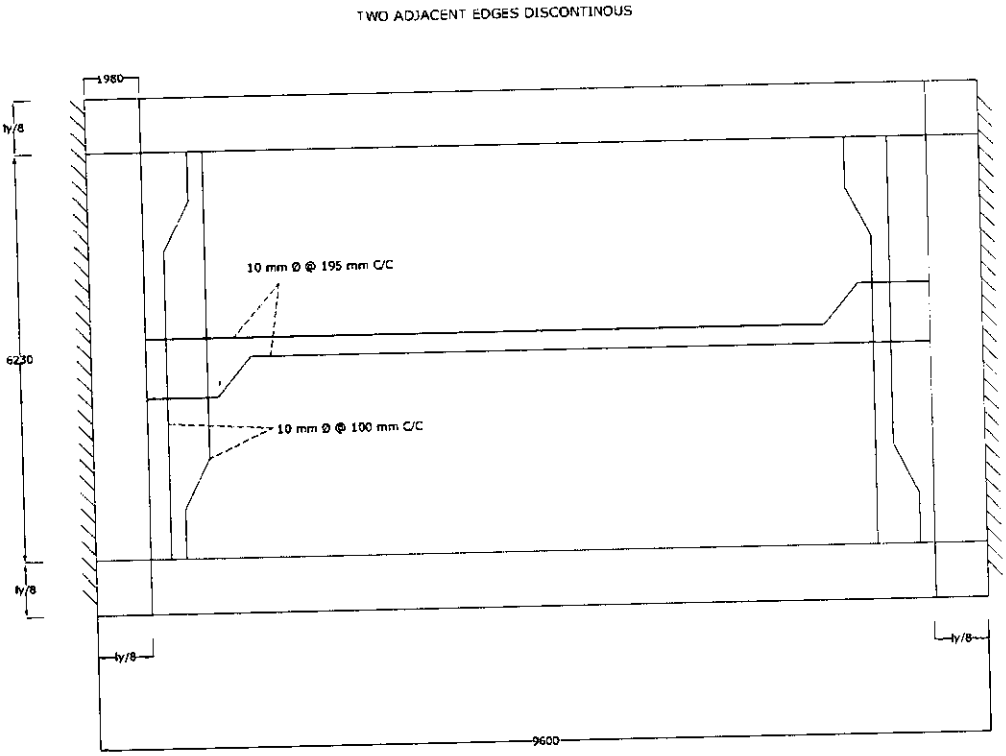
Torsion reinforcement at corners contained by two discontinuous edges

= ¾ of A_{st} max.

= 220.8mm^2

Provide 8mm @ 220mm c/c for a length equal to 1.42m*1.42m

CROSS SECTIONAL VIEW



HALL SLAB

SPECIFICATION	DESIGN	SIZE	BENDING MOMENT KN-M	REINFORCEMENT DETAILS
portico	**one way slab**	3.265*9.61	M=16.49	8 mm dia@125mm spacing Torsion: 8mm dia @300mm spacing. Size:1.9*1.9
Study room	**Two way slab**	2.03*2.93	-Mx=2.86 -My=1.89 Mx=2.14 My=1.44	8 mm dia @275 mm spacing Torsion: 6mm dia @300mm spacing Size:0.59*0.59
verandah	**Two way slab**	2.03*3.53	-Mx=3.93 -My=1.89 Mx=3.01 My=1.43	8 mm dia@275 mm spacing Torsion: 6mm dia @300mm spacing Size:0.71*0.71
Visitor's room	**Two way slab**	2.03*3.23	-Mx=3.98 -My=2.40 Mx=3.01 My=1.78	8 mm dia@275 mm spacing Torsion: 6mm dia @300mm spacing Size:0.65*0.65
Living room	**Two way slab**	9.69*6.23	-Mx=29.30 - My=nil Mx=21.99 My=16.89	10mm dia@100 mm spacing nil 10mm dia@140 mm spacing 10mm dia@195 mm spacing Torsion: 8mm dia @100mm spacing Size:1.9*1.9

Bed room+(2 toilets)	**Two way slab**	6.81*3.53	-Mx=13.72 -My=7.25 Mx=10.33 My=10.33	8 mm dia@150 mm spacing 8 mm dia@275 mm spacing 8 mm dia@200 mm spacing 8 mm dia@275 mm spacing Torsion: 8mm dia @150mm spacing Size:1.36*1.36
Bed room	**Two way slab**	3.53*3.53	-Mx=5.71 -My=5.71 Mx=4.32 My=4.32	8 mm dia @275 mm spacing Torsion: 6mm dia @300mm spacing Size:0.71*0.71
Passage & balcony	**Two way slab**	2.93*2.93	-Mx=3.93 -My=3.93 Mx=2.97 My=2.97	8 mm dia @275 mm spacing Torsion: 6mm dia @300mm spacing Size:0.59*0.59
Toilet	**Two way slab**	2.93*3.23	-Mx=5.63 -My=4.99 Mx=4.25 My=3.72	8 mm dia @275 mm spacing Torsion: 8mm dia @300mm spacing Size:0.65*0.65
Dining hall	**One way slab**	7.12*2.93	M=13.28	8 mm dia @150 mm spacing Torsion: 8mm dia @220mm spacing Size:1.42*1.42

Kitchen & drawing hall	**Two way slab**	3.23*3.23	-Mx=4.78 -My=4.78 Mx=3.61 My=3.61	8 mm dia @275 mm spacing Torsion: 6mm dia @300mm spacing Size:0.65*0.65
Pooja room	**Two way slab**	3.23*4.12	-Mx=7.23 -My=4.78 Mx=5.55 My=3.61	8 mm dia @275 mm spacing Torsion: 6mm dia @300mm spacing Size:0.82*0.82

1.5 DESIGN OF BEAM

The beam constructed at roof levels are designed as rectangular at support and T section at mid span i.e. the tension occurs at bottom of the mid span is designed as flanged beams and other portions are designed as rectangular section.

Beams are designed using limit state design procedure as Design Aids Reinforcement concrete to IS-456-2000.

BEAM A:

MOMENT AT JOINTS

A = 61505.30 N-m

B = 80856.77 N-m

C = 34096.84 N-m

D = 21281.73 N-m

MOMENT AT SPAN:

AB = 48919.93 N-m

BC = 22338.34 N-m

CD = 20480.74 N-m

DEPTH OF BEAM:

Mu/bd2 = 2.76

Assume b=230mm

Mu = 80856.77 N-m

Therefore d=360mm &

D=400mm.

AREA OF STEEL:

AT JOINT A:

$Mu = 0.87\ f_y A_{st}\, d\ (1 - (A_{st} f_y / b\ d\ f_{ck}))$

$61505.30 = 0.87\ 415\ A_{st} 360\ (1-(A_{st} 415 / 360\ 230\ 20))$

$A_{st} = 551\ mm^2$

AT JOINT B:

$Mu = 0.87\ f_y A_{st}\, d\ (1 - (A_{st} f_y / b\ d\ f_{ck}))$

$80856.77 = 0.87\ 415\ A_{st} 360\ (1-(A_{st} 415 / 360\ 230\ 20))$

$A_{st} = 770\ mm^2$

AT JOINT C:

$Mu = 0.87\ f_y\ A_{st}\, d\ (1 - (A_{st} f_y / b\ d\ f_{ck}))$

$34096.84 = 0.87\ 415\ A_{st} 360\ (1-(A_{st} 415 / 360\ 230\ 20))$

$A_{st} = 282\ mm^2$

AT JOINT D:

$Mu = 0.87\ f_y\ A_{st}\, d\ (1 - (A_{st} f_y / b\ d\ f_{ck}))$

$21281.73 = 0.87\ 415\ A_{st} 360\ (1-(A_{st} 415 / 360\ 230\ 20))$

$A_{st} = 170\ mm^2$

AT SPAN AB:

$Mu = 0.87\ f_y\ A_{st}\, d\ (1 - (A_{st} f_y / b\ d\ f_{ck}))$

$48919.93 = 0.87\ 415\ A_{st} 360\ (1-(A_{st} 415 / 360\ 230\ 20))$

$A_{st} = 426\ mm^2$

AT SPAN BC:

$Mu = 0.87\ f_y\ A_{st}\ d\ (1 - (A_{st}\ f_y\ /b\ d\ f_{ck}))$

$22338.34 = 0.87\ 415\ A_{st}\ 360\ (1-(A_{st}\ 415\ /\ 360\ 230\ 20))$

$A_{st} = 180\ mm^2$

AT SPAN CD:

$Mu = 0.87\ f_y\ A_{st}\ d\ (1 - (A_{st}\ f_y\ /b\ d\ f_{ck}))$

$20480.74 = 0.87\ 415\ A_{st}\ 360\ (1-(A_{st}\ 415\ /\ 360\ 230\ 20))$

$A_{st} = 551\ mm^2$

CHECK FOR SHEAR:

V = wl/2

V = 56084.79*4.12/2

Factored shear = 115.53 N-m

PERCENTAGE OF STEEL:

$PA = 100\ A_{st}/\ b\ d$

Pst = 0.92 %

$\acute{\Gamma}_{ve} = v/bd$

$= 1.38\ N/mm^2$

For M20 $\acute{\Gamma}_c = 2.8\ N/mm^2$

Therefore $\acute{\Gamma}_{ve} > \acute{\Gamma}_c$

Provide shear reinforcement

SPACING:

$Sv = 0.87\ f_y\ A_{st}\ d/V_{us}$

Sv = 198mm

Provide 2 legged 8mm stirrups at 198mm c/c

Anchor bars 2 no of 10mm dia to support stirrups.

CHECK FOR DELFECTION:

Actual deflection = span / effective depth

= 4120 / 360

= 11.44

Referring to chart 22

Allowable deflection = 21

Actual deflection < Allowable deflection

Hence safe.

CHECK FOR DEVELOPMENT LENGTH:

From IS-456-2000

$Ld < 1.3\ M1\ /v\ + L_0$

$1.3\ M_1\ /v\ + L0\ =\ 1297\ mm^2$

$L_d = 1197.9mm2$

$1197.9\ mm^2 < 1297\ mm^2$

Hence safe.

BEAM B:

MOMENT AT JOINTS:

A = 29191.33 N-m

B = 60336.71 N-m

C = 102777.76 N-m

D = 70908.88 N-m

MOMENT AT SPAN:

AB = 26788.03 N-m

BC = 35024.315 N-m

CD = 60462.62 N-m

DEPTH OF BEAM:

Mu/bd2 = 2.76

Assume b=230mm

M_u = 102777.76 N-m

Therefore d=400mm &

D=440mm.

AREA OF STEEL:

AT JOINT A:

$Mu = 0.87\ f_y\ A_{st}\ d\ (1 - (A_{st}\ f_y / b\ d\ f_{ck}))$

$29191.33 = 0.87\ 415\ A_{st}\ 400\ (1-(A_{st}\ 415\ /\ 400\ 230\ 20))$

$A_{st} = 212\ mm^2$

AT JOINT B:

$Mu = 0.87\ f_y\ A_{st}\ d\ (1 - (A_{st}\ f_y\ /b\ d\ f_{ck}))$

$60336.71 = 0.87\ 415\ A_{st}\ 360\ (1-(A_{st}\ 415\ /\ 400\ 230\ 20))$

$A_{st} = 466\ mm^2$

AT JOINT C:

$Mu = 0.87\ f_y\ A_{st}\ d\ (1 - (A_{st}\ f_y\ /b\ d\ f_{ck}))$

$102777.76 = 0.87\ 415\ A_{st}\ 360\ (1-(A_{st}\ 415\ /\ 400\ 230\ 20))$

$A_{st} = 890\ mm^2$

AT JOINT D:

$Mu = 0.87\ f_y\ A_{st}\ d\ (1 - (A_{st}\ f_y\ /b\ d\ f_{ck}))$

$70908.88 = 0.87\ 415\ A_{st}\ 360\ (1-(A_{st}\ 415\ /\ 400\ 230\ 20))$

$A_{st} = 521\ mm^2$

AT SPAN AB:

$Mu = 0.87\ f_y\ A_{st}\ d\ (1 - (A_{st}\ f_y\ /b\ d\ f_{ck}))$

$26788.03 = 0.87\ 415\ A_{st}\ 360\ (1-(A_{st}\ 415\ /\ 400\ 230\ 20))$

$A_{st} = 193\ mm^2$

AT SPAN BC:

$Mu = 0.87\ f_y\ A_{st}\ d\ (1 - (A_{st}\ f_y\ /b\ d\ f_{ck}))$

$35024.315 = 0.87\ 415\ A_{st}\ 360\ (1-(A_{st}\ 415\ /\ 400\ 230\ 20))$

$A_{st} = 257\ mm^2$

AT SPAN CD:

$Mu = 0.87\ f_y\ A_{st}\ d\ (1 - (A_{st}\ f_y / b\ d\ f_{ck}))$

$60462.62 = 0.87\ 415\ A_{st}\ 360\ (1-(A_{st}\ 415\ /\ 400\ 230\ 20))$

$A_{st} = 468\ mm^2$

CHECK FOR SHEAR:

$V = wl/2$

$V = 112077 * 3.23/2$

Factored shear = 181 KN

PERCENTAGE OF STEEL:

$P_{st} = 100\ A_{st} / b\ d$

$P_{st} = 0.86\ \%$

$\acute{\Gamma}_{ve} = v/bd$

$= 1.96\ N/\ mm^2$

For M20 $\acute{\Gamma}_c = 2.8\ N/\ mm^2$

Therefore $\acute{\Gamma}_{ve} > \acute{\Gamma}_c$

Provide shear reinforcement

SPACING:

$S_v = 0.87\ f_y\ A_{st}\ d / V_{us}$

$S_v = 110mm$

Provide 2 legged 8mm stirrups at 110mm c/c

Anchor bars 2 no of 10mm dia to support stirrups.

CHECK FOR DELFECTION:

Actual deflection = span / effective depth

= 3230 / 400

= 8.075

Referring to chart 22

Allowable deflection = 21

Actual deflection < Allowable deflection

Hence safe.

CHECK FOR DEVELOPMENT LENGTH:

From IS-456-2000

$Ld < 1.3\ M1\ /v\ + L_0$

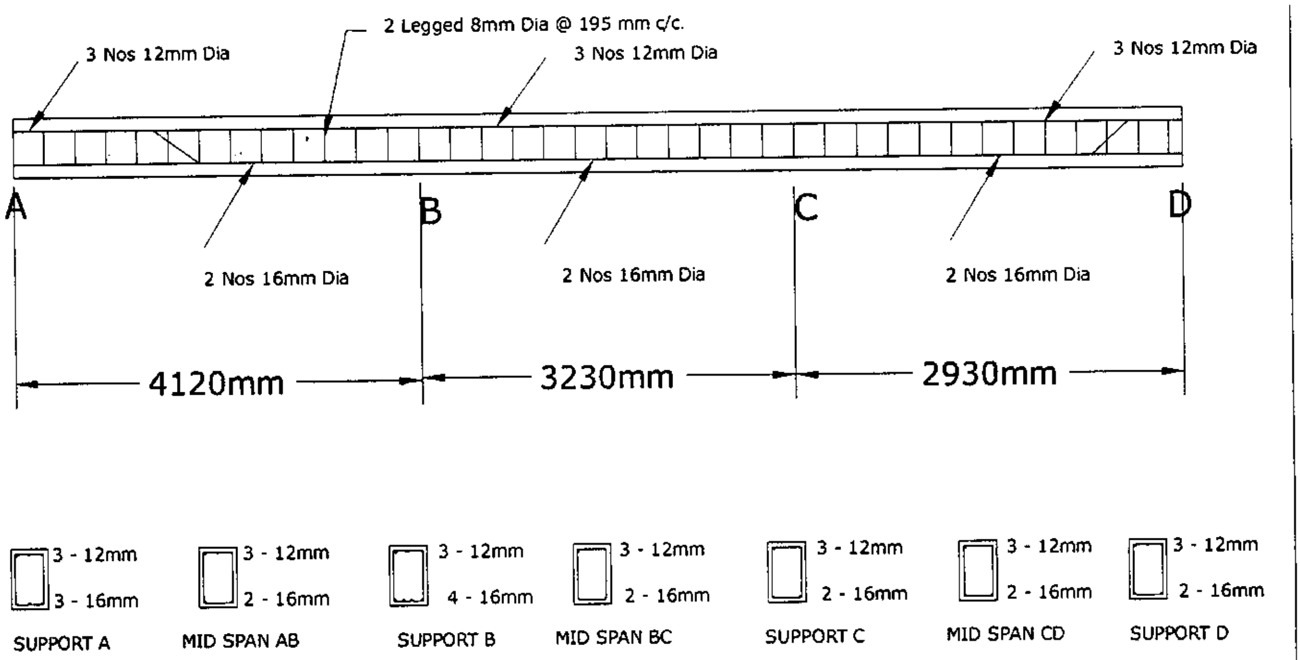
$1.3\ M_1\ /v\ + L_0 = 1401.621mm^2$

$Ld = 1197.9mm^2$

$1197.9mm^2 < 1401.621mm^2$

Hence safe.

REINFORCEMENT DETAILS OF BEAM A:



1.6 DESIGN OF FOOTING

COLUMN FOOTING:

Individual footing provided to distribute the load of a column to the soil is called isolated footing.

This footing can be

Square for square or circular column

Rectangular for rectangular column

Circular for circular column

Let

W = load on the column footing from column

W1 = self weight of footing, generally taken as 10 to 20 % of w

P0 = bearing capacity of soil

Area of footing can be calculated from the relation

$$A = (W+W1)/p_0$$

Provide this much area in a form depending upon the type of the column

i. Square footing $B = \sqrt{A}$

ii. Rectangular footing $B*L = A$

iii. Circular footing $3.14\ D^2 = A$

DESIGN OF FOOTING:

Axial load = 195.83 KN

Self weight = 19.5 KN

(10% of axial load)

Total load =215.83 KN

Assume total load =220 KN

Factored axial load $=1.5\times220$

$=330$ KN

Area =(Load/safe bearing capacity)

=(330/230)

$=1.43\ m^2$

Provide footing size of 1.2m×1.2m

Reaction =(Total load/Area)

=(330/1.44)

$=229.17\ KN/m^2$

TO FIND DEPTH:

ONE WAY SHEAR:

$$D = (p(L - b)/(2p + 700L^2))$$

$$= 330 \times (1.2 - 0.23)/((2 \times 330)+(700 \times 1.2^2))$$

$$= 0.191\ m$$

Provide d=200 mm

Increase the depth three times as per codal provisions

Hence d =600 mm

FOR TWO WAY SHEAR:

Perimeter=4 (B + d)

=4 (0.23+0.6)

Perimeter =3.32 m

Shear $=229.17\times(3.32^2-0.83^2)$

=2368.13 KN

Shear stress $=0.25\times\sqrt{f_{ck}}$

Assume $f_{ck}=20\ N/mm^2$

shear stress x perimeter x depth = shear

$1.118\times3.32\times d=2368.13$

$d=610$ mm

Hence, provide $d=600$ mm

TO FIND MOMENT:

Moment$=p(L-b)^2/8L$

$=330\times(1.2-0.23)^2/(8\times1.2)$

$=32.34$ KN-m

From annex G,

$m_u=0.87\times fy\times Ast\times d\times(1-(fy\times A_{st})/(b\times d\times f_{ck}))$

$A_{st}=150\ mm^2$

As area of steel is very less, provide minimum reinforcement

Min $A_{st}=(0.12/100)\times1000\times650$

$=780\ mm^2$

Assume 16 mm dia bar

Number of bars $=(A_{st}/\text{Area of one bar})$

$=4$

Spacing $=(\text{Area of one bar}/A_{st})$

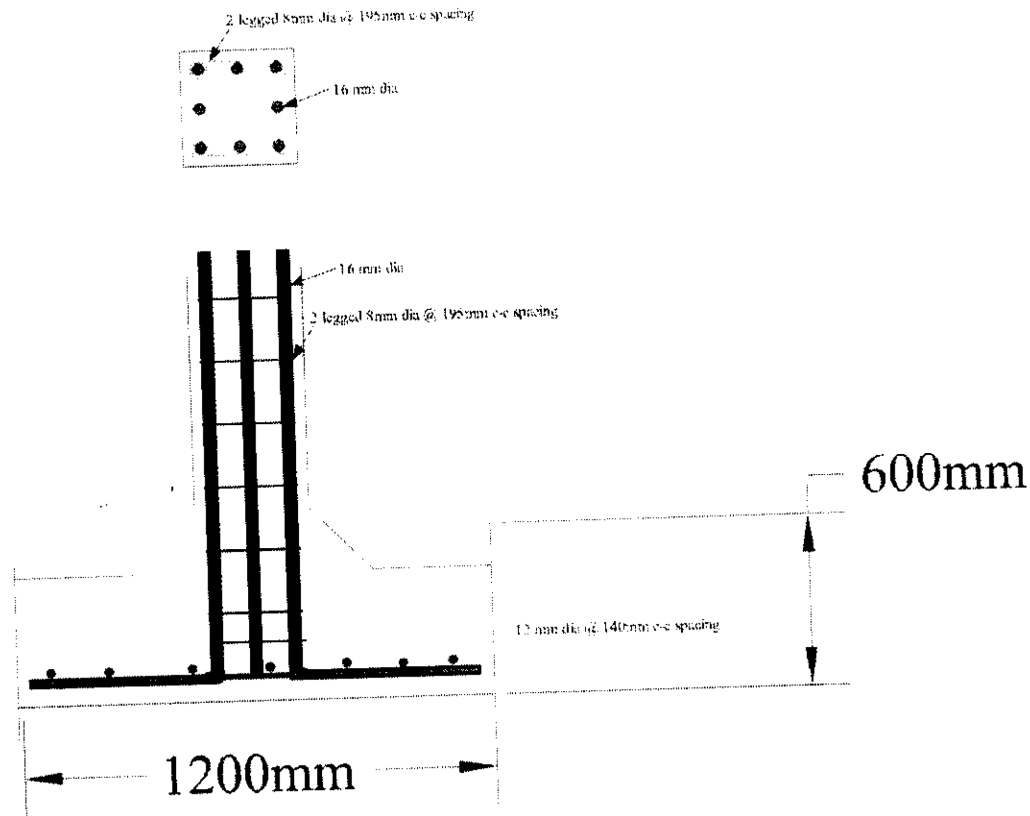
$=250$ mm

Provide 4 rods of 16 mm dia @250 mm c/c

Total depth = 600 + 50 + (16/2)

= 658 mm

REINFORCEMENT DETAILS OF FOOTING:



1.7 DESIGN OF COLUMN

INDRODUCTION:

Columns are vertical compression members used to transfer the loads of the structures such as buildings, factory floors, cinema balconies, auditorium hall, floors of framed buildings, etc., to the foundation below.

The transfer of load may be

1. Direct from the roof or floor slabs through the columns to the foundation.
2. Indirect through a beam to the column and then to the foundation.

All vertical members may not be termed as columns. Only those members whose effective length as more than three times the least lateral dimension are called columns and those members whose effective length is less than 3 times the least lateral dimension are called pedestals.

Axially loaded columns are those in which the line of action of resultant thrust of the load supported by a column coincides with the centre of gravity of the column cross section. As per IS-456-1978 there is no column which is truly axially loaded. It says that all columns shall be designed for minimum eccentricity equal to the unsupported length of column/500 plus lateral dimension/30, subject to a minimum of 20mm.

TYPES OF COLUMNS:

The various types pf R.C. columns are

- Columns with longitudinal steel and with lateral ties or spirals
- Composite columns with structural Rolled steel section encased in concrete.
- ` Concrete filled steel tubular columns in which steel tube filled with concrete inside it.

SHORT AND SLENDER COLUMNS:

A compression member may be considered as short when both the slenderness ratios l_{ex}/D and l_{ey}/b are less than 12

Where l_{ex}, l_{ey} = effective length in respect of the major axis and minor axis respectively.

D = depth in respect of the major axis.

B = width of the member.

If the above slenderness ratios are greater than 12, then it shall be considered as slender as slender compression member or a long column.

DESIGN OF COLUMN:

Axial load =330 KN

X = 0.23

Y = 0.23

SLENDER CHECK:

$(l_{ex}/b) = 13.04 > 12$(long column)

$(l_{ey}/b) = 13.04 > 12$(long column)

Design both x& y as long column

From table 1, Design Aids

$(l_{ex}/D) = 13.04$ $(e_{ax}/D) = 0.085$

$(l_{ey}/b) = 13.04$ $(e_{ay}/b) = 0.086$

Additional moments

$Max = p_u . e_x$

$= 6.45$ KN-m.

$May = p_u . e_y$

$= 6.52$ KN-m.

The above moments should be reduced in accordance with 38.7.1.1 of code but multiplication factors can be evaluated only if reinforcement is known

Trial assume p=3%

$A_g = 23 * 23 = 529cm^2$

From chart 63

$(p_{uz}/Ag) = 18N/mm^2$

$P_{uz} = 952.2$ KN

CALCULATION OF P_b:

Assuming 25 mm bar with 40 mm cover

d'/D(xx axis) = 0.22

Chart or table for d'/D = 0.2 is used.

d'/D(yy axis)=0.22

Chart or table for d'/D = 0.2 is used

$p_{bx} = (k1 + k2p/f_{ck})\, f_{ck}\, b\, D$

$k1 = 0.184$

$k2 = 0.0208$

$p_{bx} = 330.1$ KN

$p_{by} = (k1 + k2p/f_{ck}) f_{ck}\, b\, D$

$p_{by} = 330.1$ KN

$K_x = (p_{uz} - p_u)/(p_{uz} - p_{bx})$

$K_x = 1$

$K_y = 1$

Additional moments

$M_{ax} = 6.45*1 = 6.45$ KN-m

$M_{ay} = 6.52*1 = 6.52$ KN-m

Additional moments due to slenderness ratio should be added to initial value

$M_{ux} = (0.6*23 - 0.4*10)$

$M_{ux} = 9.8$ KN-m

$M_{uy} = 5.8$ KN-m

$e_x = (l/500+D/30)$

$e_x = 1.36$ KN-m

$e_y = (l/500+D/30)$

$e_y = 1.36$ KN-m

Both e_x & e_y are less than 2

Hence no need to consider this.

$M_{ux} = 6.45$ KN-m

$M_{uy} = 6.52$ KN-m

$(p_u/f_{ck}\, b\, D) = 0.311$

$(p_u/f_{ck}) = (3/20) = 0.15$

Reffering chart 45

$(M_u/f_{ck} b\, d^2) = 0.16$

$M_{ux1} = 38.9$ KN-m

Reffering chart 45

$(M_u/f_{ck} b\, D^2) = 0.16$

$M_{uy1} = 38.9$ KN-m

$(M_{ux} / M_{ux1}) = 0.17$

$(M_{uy} / M_{uy1}) = 0.17$

$(p_u/p_{uz}) = (330/952.2) = 0.346$

$\alpha^n = 0.98$

CHECK:

$(M_{ux}/ M_{ux1})^{\alpha n} + (M_{uy}/ M_{uy1})^{\alpha n} < 1$

$(0.17)\ 0.98 + (0.17)\ 0.98 = 0.35 < 1$

Hence section is ok

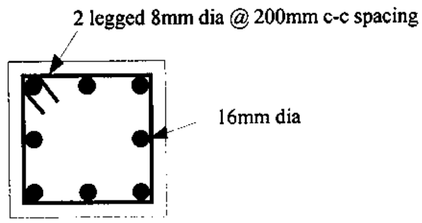
$A_s = (\ p\ b\ D / 100\ )$

$A_s = 1587\ mm^2$

Assume 25 mm bar

Provide 4 bars of 25 mm dia @300 mm c/c.

CROSS SECTIONAL VIEW



REBAR DETAILS OF COLUMN

1.8 STEEL DESIGN

DESIGN OF BEAM:

Span moment(max),B.M = 80.86KNm

M = f * Z

f = 0.66 * fy

fy = 250N/mm2

Zreq = M / f

= 490.060*103 mm3

Assuming ISMB300

Zprov = 573.6*103 mm3

M/Zprov = 141.12 < 0.66fy

Hence the assumed section is safe.

DESIGN OF COLUMN:

Axial load Pu = 330KN

Assume slenderness ratio as 90

σ_{ac} = 90Mpa (for IS800- 1984 , pg39)

= 90N/mm2

Required area of section = load / σ_{ac}

= 330*103 / 90

= 3666.67 mm2

Select an I section from steel table for required area

Let us provite ISWB200 section

$r_{xx} = 84.6mm$

$r_{yy} = 29.9mm$ A= 3671mm2

$r_{min} = 29.9mm$

with end condition pg41 table 5.2(c)

effective length, l = 1.0L

= 1*3*103

= 3000mm

$\lambda = l / r_{min}$

= 3000/29.9

= 100.33

Since λ is greater than assumed λ so select ISHB150 (pg.4) steel tables

$r_{xx} = 62.9mm$

$r_{yy} = 34.4mm$ A=3898mm2

$r_{min} = 34.4mm$

$\lambda = l / r_{min}$

= 3000/34.4

= 87.21

80 101

87.21→

90 90

$\sigma ac = 101 - [(87.21-80)/(90-80)] * (101-90)$

$= 101-7.931$

$= 93.069$ Mpa

$= 93.069$ N/mm2

Safe load = σac * A

$= 93.069 * 3898$

$= 362782.96$N

$= 362.78$KN

Since safe load calculated is greater than applied load .

The selected design is safe .

DESIGN OF SLAB BASE:

Axial load = 330KN

Allowable bearing pressure on concrete = 4N/mm2

Bending stress on slab base = 185 N/mm2

Column section = ISWB150

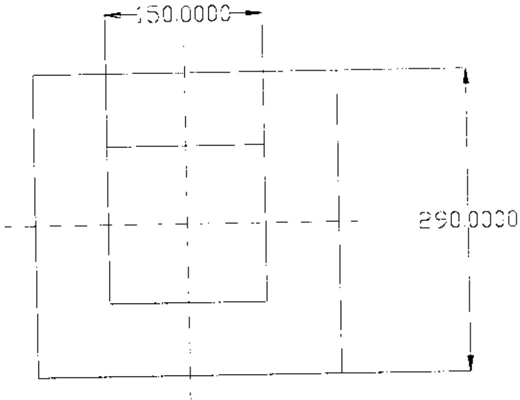
Design:

Area of slab base = (330*1000)/4

= 8.25*104mm2

Size of column section ISHB150 = 150*150mm2

Area of slab base = (150+2a)(150+2b)mm2



SLAB BASE(PLAN)

Projections:

Let projections a@b be equal

Area of slab (150+2a)2 = 8.25*104

150+2a = 287.23

a = 68.61mm

provide projections a=b = 70mm

provide slab base = (150+2*70)2

= 84100mm2

Intensity of pressure

W = (330*1000)/84100

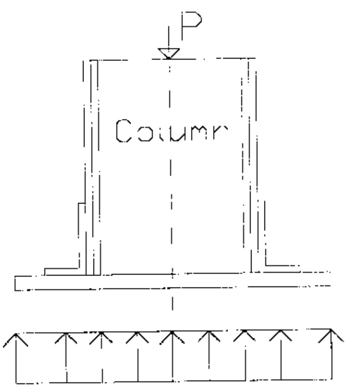
= 3.92N/mm2

Thickness of slab base:

Thickness = [((3*3.92)/185)(702(702/4))]1/2

= 15.34mm

Provide 16mm thick slab base.



PRESSURE DISTRIBUTION
SLAB BASE

DESIGN OF CONNECTIONS:

End reaction 330KN = F

Column ISHB150

BEAM ISMB300

For beam:

Width of flange 140mm = bf

Thickness of web tw = 7.5mm

Thickness of flange tf = 12.4mm

B = 125mm

h2 = 29.25mm

bearing length required

$$b = [(F/(tb.tw))-\sqrt{3}h2)]$$

$$= \{[(330*1000)/(185*7.5)]-(\sqrt{3}*29.25)\}$$

$$= 140.12mm$$

$$b > ((.5*330)/(185*7.5))$$

$$> 118.92mm$$

Bearing length = 140.12mm

10mm clearance width of seat plate = 150.12mm

Adopt width of seat plate = 160mm

Thickness of flange beam = 12.4mm

Provide 13mm thick , 160mm long & 140mm wide seat plate

Thickness of stiffening plate = thickness of web of beam

= 7.5mm

Provide 10mm thick stiffening plate

Distance of end reaction from outer end of seat plate

$$(160-(0.5*140.12)) = 89.94mm$$

Bending moment = M = (330*89.94)

= 26982KNm

Properties of welds:

$[(2*250*125)/(2*250)+(130)] = 99.21mm$

$Y_1 = 99.21mm$

$Y_2 = 150.79mm$

$Ixx = 4007*10^4 mm4$

Stress in welds:

$\tau_{vsv} = [(330*1000)/(2*250)]$

$= 600N/mm$

$\tau_{vsh} = [(26982*1000*472)/(959.344*104)]$

$= 667.98N/mm$

Resultant shear stress Fs = (6002+6672)1/2

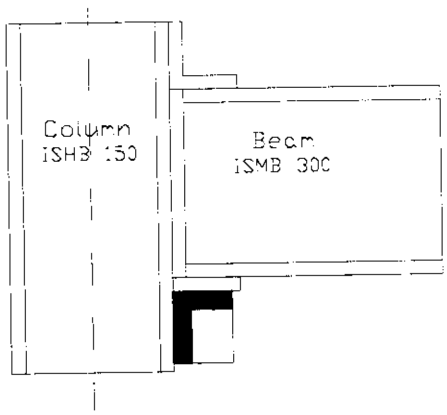
= 897.88 N/mm

Size of weld

$0.7*s*1mm*10 = 897.88$

$s = 11.66mm$

provide 12mm fillet welds ISA100*100*6mm is used at top.



STIFFENED WELDED SEAT CONNECTIONS

1.9 STAAD PRO RESULTS – STEEL STRUCTURE

```
ALL UNITS ARE - KN   METE (UNLESS OTHERWISE NOTED)

MEMBER    TABLE      RESULT/  CRITICAL COND/   RATIO/    LOADING/
            FX        MY        MZ     LOCATION
=======================================================================

    1 ST  ISHB150          (INDIAN SECTIONS)
              PASS    IS-7.1.1(B)     0.763       3
           206.12 C     1.96       4.69      2.10
    2 ST  ISHB150          (INDIAN SECTIONS)
              PASS    IS-7.1.1(A)     0.992       3
           341.94 C     1.68      -1.56      2.10
    3 ST  ISHB150          (INDIAN SECTIONS)
              PASS    IS-7.1.1(A)     0.970       3
           349.37 C     1.18       1.74      2.10
*   4 ST  ISHB150          (INDIAN SECTIONS)
              FAIL    IS-7.1.1(B)     1.703       3
           311.03 C     8.54      -6.01      2.10
    5 ST  ISHB150          (INDIAN SECTIONS)
              PASS    IS-7.1.1(A)     0.974       3
           367.19 C     0.05       4.60      2.10
*   6 ST  ISHB150          (INDIAN SECTIONS)
              FAIL    IS-7.1.1(A)     1.100       3
           402.78 C     0.30       5.33      2.10
*   7 ST  ISHB150          (INDIAN SECTIONS)
              FAIL    IS-7.1.1(A)     1.697       3
           485.06 C     4.19       4.22      2.10
*   8 ST  ISHB150          (INDIAN SECTIONS)
              FAIL    IS-7.1.1(A)     1.461       3
           388.48 C    -4.85       3.71      2.10
    9 ST  ISHB150          (INDIAN SECTIONS)
              PASS    IS-7.1.1(A)     0.348       3
            57.50 C     0.47       4.83      2.10
```

```
ALL UNITS ARE - KN   METE (UNLESS OTHERWISE NOTED)

MEMBER    TABLE      RESULT/  CRITICAL COND/   RATIO/    LOADING/
            FX        MY        MZ     LOCATION
=======================================================================

*  10 ST  ISHB150          (INDIAN SECTIONS)
              FAIL    IS-7.1.1(A)     1.334       3
           527.59 C    -0.65      -0.53      2.10
*  11 ST  ISHB150          (INDIAN SECTIONS)
```

```
                FAIL    IS-7.1.1(A)      1.437       3
             499.06 C       1.30       -4.93       2.10
*   12 ST   ISHB150              (INDIAN SECTIONS)
                FAIL    IS-7.1.1(A)      1.612       3
             462.05 C      -4.62       -1.61       2.10
    13 ST   ISHB150              (INDIAN SECTIONS)
                PASS    IS-7.1.1(A)      0.779       3
             291.44 C       0.72       -1.37       2.10
    14 ST   ISHB150              (INDIAN SECTIONS)
                PASS    IS-7.1.1(A)      0.619       3
             220.06 C      -1.09       -0.71       2.10
*   15 ST   ISHB150              (INDIAN SECTIONS)
                FAIL    IS-7.1.1(A)      1.173       3
             447.10 C       0.39       -3.37       2.10
    16 ST   ISHB150              (INDIAN SECTIONS)
                PASS    IS-7.1.1(A)      0.979       3
             385.98 C      -0.70        0.07       2.10
*   17 ST   ISHB150              (INDIAN SECTIONS)
                FAIL    IS-7.1.1(A)      1.313       3
             442.46 C      -2.21        1.90       2.10
    18 ST   ISHB150              (INDIAN SECTIONS)
                PASS    IS-7.1.1(A)      0.816       3
             309.28 C       0.52        1.86       2.10


ALL UNITS ARE - KN   METE (UNLESS OTHERWISE NOTED)

MEMBER    TABLE       RESULT/  CRITICAL COND/   RATIO/    LOADING/
            FX          MY         MZ     LOCATION
=======================================================================


    19 ST   ISHB150              (INDIAN SECTIONS)
                PASS    IS-7.1.1(A)      0.283       3
             117.75 C       0.03        0.15       2.10
*   20 ST   ISHB150              (INDIAN SECTIONS)
                FAIL    IS-7.1.1(A)      1.731       3
             481.89 C       4.34       -5.35       2.10
*   21 ST   ISHB150              (INDIAN SECTIONS)
                FAIL    IS-7.1.1(A)      1.579       3
             398.79 C      -5.52       -4.72       2.10
    22 ST   ISHB150              (INDIAN SECTIONS)
                PASS    IS-7.1.1(B)      0.314       3
              65.98 C      -0.03       -5.41       2.10
    23 ST   ISHB150              (INDIAN SECTIONS)
                PASS    IS-7.1.1(A)      0.694       3
             233.02 C      -1.29        1.97       2.10
    24 ST   ISHB150              (INDIAN SECTIONS)
                PASS    IS-7.1.1(B)      0.498       3
             124.24 C       0.05        7.47       2.10
*   25 ST   ISHB150              (INDIAN SECTIONS)
                FAIL    IS-7.1.1(A)      1.847       3
             492.88 C      -4.76       -6.96       2.10
    26 ST   ISLB250              (INDIAN SECTIONS)
                PASS    IS-7.1.2         0.423       3
```

	0.93 T	0.04	20.28	2.93
27 ST	ISLB250		(INDIAN SECTIONS)	
	PASS	IS-7.1.2	0.442	3
	2.88 T	-0.07	20.74	3.23

STEEL TAKE-OFF

PROFILE	LENGTH(METE)	WEIGHT(KN)
ST ISHB350	478.68	315.870
	TOTAL =	315.870

MEMBER	PROFILE	LENGTH (METE)	WEIGHT (KN)
1	ST ISHB350	2.10	1.386
2	ST ISHB350	2.10	1.386
3	ST ISHB350	2.10	1.386
4	ST ISHB350	2.10	1.386
5	ST ISHB350	2.10	1.386
6	ST ISHB350	2.10	1.386
7	ST ISHB350	2.10	1.386
8	ST ISHB350	2.10	1.386
9	ST ISHB350	2.10	1.386
10	ST ISHB350	2.10	1.386
11	ST ISHB350	2.10	1.386
12	ST ISHB350	2.10	1.386
13	ST ISHB350	2.10	1.386
14	ST ISHB350	2.10	1.386
15	ST ISHB350	2.10	1.386
16	ST ISHB350	2.10	1.386
17	ST ISHB350	2.10	1.386
18	ST ISHB350	2.10	1.386
19	ST ISHB350	2.10	1.386
20	ST ISHB350	2.10	1.386
21	ST ISHB350	2.10	1.386
22	ST ISHB350	2.10	1.386
23	ST ISHB350	2.10	1.386
24	ST ISHB350	2.10	1.386
25	ST ISHB350	2.10	1.386
26	ST ISHB350	2.93	1.933
27	ST ISHB350	3.23	2.131
28	ST ISHB350	3.23	2.131
29	ST ISHB350	4.12	2.719
30	ST ISHB350	6.23	4.111
31	ST ISHB350	2.03	1.340
32	ST ISHB350	3.23	2.131
33	ST ISHB350	3.53	2.329
34	ST ISHB350	3.29	2.171
35	ST ISHB350	6.23	4.111
36	ST ISHB350	2.03	1.340
37	ST ISHB350	3.53	2.329
38	ST ISHB350	5.26	3.471

39	ST ISHB350	1.73	1.142
40	ST ISHB350	3.53	2.329
41	ST ISHB350	4.06	2.679
42	ST ISHB350	3.23	2.131
43	ST ISHB350	3.23	2.131
44	ST ISHB350	3.23	2.131

45	ST ISHB350	3.53	2.329
46	ST ISHB350	3.23	2.131
47	ST ISHB350	3.53	2.329
48	ST ISHB350	2.93	1.933
49	ST ISHB350	2.93	1.933
50	ST ISHB350	2.93	1.933
51	ST ISHB350	2.93	1.933
52	ST ISHB350	3.23	2.131
53	ST ISHB350	4.12	2.719
54	ST ISHB350	3.29	2.171
55	ST ISHB350	2.93	1.933
56	ST ISHB350	2.93	1.933
57	ST ISHB350	2.93	1.933
58	ST ISHB350	2.03	1.340
59	ST ISHB350	2.03	1.340
67	ST ISHB350	3.00	1.980
68	ST ISHB350	3.00	1.980
72	ST ISHB350	3.00	1.980
73	ST ISHB350	3.00	1.980
77	ST ISHB350	3.00	1.980
80	ST ISHB350	3.00	1.980
81	ST ISHB350	3.00	1.980
82	ST ISHB350	3.00	1.980
85	ST ISHB350	2.93	1.933
86	ST ISHB350	3.23	2.131
87	ST ISHB350	3.23	2.131
88	ST ISHB350	4.12	2.719
89	ST ISHB350	6.23	4.111
90	ST ISHB350	2.03	1.340
91	ST ISHB350	3.23	2.131
92	ST ISHB350	3.53	2.329
93	ST ISHB350	3.29	2.171
94	ST ISHB350	6.23	4.111
95	ST ISHB350	2.03	1.340
96	ST ISHB350	3.53	2.329
97	ST ISHB350	5.26	3.471
98	ST ISHB350	1.73	1.142
99	ST ISHB350	3.53	2.329
100	ST ISHB350	4.06	2.679
101	ST ISHB350	3.23	2.131
102	ST ISHB350	3.23	2.131
103	ST ISHB350	3.23	2.131
104	ST ISHB350	3.53	2.329
105	ST ISHB350	3.23	2.131
106	ST ISHB350	3.53	2.329
107	ST ISHB350	2.93	1.933
108	ST ISHB350	2.93	1.933
109	ST ISHB350	2.93	1.933

110	ST ISHB350	2.93	1.933
111	ST ISHB350	3.23	2.131
112	ST ISHB350	4.12	2.719
113	ST ISHB350	3.29	2.171
114	ST ISHB350	2.93	1.933
115	ST ISHB350	2.93	1.933
116	ST ISHB350	2.93	1.933
117	ST ISHB350	2.03	1.340

118	ST ISHB350	2.03	1.340
136	ST ISHB350	3.00	1.980
137	ST ISHB350	3.00	1.980
138	ST ISHB350	3.00	1.980
139	ST ISHB350	3.00	1.980
140	ST ISHB350	3.00	1.980
141	ST ISHB350	3.00	1.980
142	ST ISHB350	3.00	1.980
143	ST ISHB350	3.00	1.980
144	ST ISHB350	3.00	1.980
145	ST ISHB350	3.00	1.980
146	ST ISHB350	3.00	1.980
147	ST ISHB350	3.00	1.980
148	ST ISHB350	3.00	1.980
149	ST ISHB350	3.00	1.980
150	ST ISHB350	3.00	1.980
151	ST ISHB350	3.00	1.980
152	ST ISHB350	3.00	1.980
153	ST ISHB350	2.93	1.933
154	ST ISHB350	3.23	2.131
155	ST ISHB350	3.23	2.131
156	ST ISHB350	4.12	2.719
157	ST ISHB350	3.53	2.329
158	ST ISHB350	3.29	2.171
159	ST ISHB350	5.26	3.471
160	ST ISHB350	1.73	1.142
161	ST ISHB350	3.53	2.329
162	ST ISHB350	4.06	2.679
163	ST ISHB350	3.23	2.131
164	ST ISHB350	3.23	2.131
165	ST ISHB350	3.23	2.131
166	ST ISHB350	3.53	2.329
167	ST ISHB350	2.93	1.933
168	ST ISHB350	2.93	1.933
169	ST ISHB350	2.93	1.933
170	ST ISHB350	2.93	1.933
171	ST ISHB350	3.23	2.131
172	ST ISHB350	4.12	2.719
173	ST ISHB350	3.29	2.171
174	ST ISHB350	2.93	1.933
175	ST ISHB350	3.00	1.980
176	ST ISHB350	3.00	1.980
177	ST ISHB350	3.00	1.980
178	ST ISHB350	3.00	1.980
179	ST ISHB350	3.00	1.980
180	ST ISHB350	3.00	1.980

181	ST ISHB350	3.00	1.980
182	ST ISHB350	3.00	1.980
183	ST ISHB350	3.00	1.980
184	ST ISHB350	3.00	1.980
185	ST ISHB350	3.00	1.980
186	ST ISHB350	3.00	1.980
187	ST ISHB350	3.00	1.980
188	ST ISHB350	3.00	1.980
189	ST ISHB350	3.00	1.980
190	ST ISHB350	3.00	1.980
191	ST ISHB350	3.00	1.980

TOTAL = 315.870

************ END OF DATA FROM INTERNAL STORAGE ************

1.10 COMPARISON BETWEEN STEEL AND CONCRETE

Which is more sustainable - concrete or steel-framed buildings? This is a question people have asked themselves time and time again and, as a result, many comparisons have been made. Owners and project teams must work together to evaluate every aspect of a proposed project to deliver "the most bang for the buck." One key decision before embarking on the design of a new project is determining the type of building structure to use. Residential building structures are commonly built with either structural steel or reinforced concrete. So which option is best, steel or concrete? Depending on the specifics of a particular project, both! Owners must consider several factors before this question can be properly answered. This project seeks to compare a steel-framed residential building using composite floor construction – concrete floors supported on steel beams – with, an alternative of a concrete frame with in situ concrete floors. Comparison can be made only using certain parameters. The parameters for comparison are as follows

- Property wise comparison
- Cost wise comparison
- Environmental consideration

Property wise comparison:

With world demand growing for housing that is needed sooner rather than later to meet the need, steel is increasing in popularity as the choice for such constructions as residential steel buildings. Built from light-gauge steel, they are affordable and easy to complete on site after initial assembly of components at a factory.

- Strength is a major plus for steel. Steel structures can withstand unfavorable weather conditions such as hurricanes, high winds, heavy snow and even earthquakes.
- The usage of steel results in lighter structures with stronger connections which eventually have lesser seismic force.
- Steel is stable and hence, weather conditions such as moisture will not cause it to contract or expand. When planning steel residential modular buildings, there is no need to worry that the steel will rot, warp, or crack.
- Steel is lighter than other framing materials and is fire resistant which makes it advantageous over other materials like wood, which are combustible. The quality of steel is consistent.
- Moreover, steel is 66% recyclable, which makes it an especially cost-effective and environmentally sound alternative to any other construction material.
- Steel when used for foundations accounts for very minor movements which is a major advantage as it does not cause any foundation problems.
- Steel has the highest strength to weight ratio of all the other comparable building materials. Section size, for a given span and loading, a steel section will take up less space.
- Low wastage. Because of the ability to weld steel sections, small lengths are not thrown away. Most welding shops have piles of offcuts that are regularly picked through for small jobs or for adding to longer lengths
- In certain locations, geographic (as in near the coast), or in the house (as in bathrooms and other wet areas), there is need for extra corrosion protection over and above the normal.

- Thermal expansion and contraction in some surfaces, especially sprung curved roofs causes loud movement noises. Similarly heavy rain noise can be intrusive.
- Thermal insulation. Steel is a good conductor of heat or cold, and as such extra measures have to be taken to insulate the residence.
- **Loading.** Both concrete and steel frame buildings can safely support the load, but designing a steel frame building to support such increased loads could come at a premium. Furthermore, consideration must be given to the protective shielding requirements of the imaging equipment, such as increased wall thickness, which will most likely have structural implications.
- **Vibration.** This is an important consideration when the facility will house equipment and procedures that are particularly sensitive. A concrete frame structure is inherently more capable of minimizing both vibration and noise. Steel frame structures can be designed to minimize vibration; however, as with loading, such measures can be costly.
- **Seismic Classification.** Early in the design phase, the structural engineer will require soils/seismic reports to help determine the existing site conditions and how they could impact the building structure. The resulting Seismic Design Category, indicated in a range from "A" to "D" (with "A" being most favorable), will impact the type of structure that should be used. An "A" seismic classification gives an engineer the greatest flexibility to provide a safe design while eliminating unnecessary cost.

Cost wise comparison:

Initial Cost

One critical factor when determining whether to use a steel or reinforced concrete structure is the total impact on building cost. The rising cost of building materials has been well documented in recent years, with both steel and concrete susceptible to increases. The most notable material price increase has been in structural steel, though concrete cost can be affected by factors such as cement production and petroleum prices. Increases in petroleum costs have an effect on the manufacturing and transportation of most materials, but the impact may be greatest on ready-mix concrete, which requires numerous trips to the jobsite. However, the overall impact on project cost cannot be measured solely by the cost of raw material; there are several other factors to consider, the first of which is labor. The availability of a qualified labor force can significantly impact both cost and schedule. The labor force issue is most pronounced with concrete, as concrete placement requires more on-site production labor. Finishing is another factor that can further affect cost. For example, a steel frame requires sprayed fireproofing (concrete is inherently more fireproof) and additional labor from other trades such as drywall, mechanical and electrical. Because factors such as material pricing and available labor force are both time and market sensitive, owners must evaluate initial steel and concrete cost on a pre-project basis.

Schedule Impact

It is often said that time is money. Many experts in the construction industry would argue that a structural steel building can be built faster. This is often true. But as with cost evaluation, it is important to consider the pros and cons inherent to each structural system and how those factors will impact the project schedule. The long lead time for steel has become a critical issue when considering this option. Steel manufacturers have tightened mill production, which could require a Construction Manager to engage both a steel detailer and a fabricator early in the job to ensure that critical steel components are ordered in time. This technique can be effective, but it requires the owner to make early design and cash commitments to reserve steel. In order to take advantage of quick structural steel erection, the steel must be ordered early enough to avoid a potential delay waiting on materials. Conversely, a concrete structure can begin quickly and with minimal lead time. However, because a concrete structure is built entirely onsite, it requires more complete structural documents to begin construction, as well as more manpower and work hours to complete .As with initial cost, schedule impact for both steel and concrete must be evaluated on a pre-project basis. The current availability of raw materials and the labor force will have a significant impact on how quickly a project can be built.

Various cost saving benefits are involved in the construction of residential steel buildings. Because a lot of the work has already been done in the factory before the final on-site construction gets underway, you will save a lot of labor costs. The team you use to do the final work does not have to be the most expensive one in town, because of the relative ease of the required work. As well as being easy, construction of residential modular buildings is quick, compared with the time taken on other types of buildings. As well as saving on labor costs, therefore, you will also save on time. Even with foundation costs and whatever extras you pay for custom design, you will find residential steel buildings are kinder to your budget than other styles of homes. With steel buildings, your ongoing costs should therefore be lower than with other types of houses.

Environmental consideration:

Steel

It is estimated that, worldwide, more than 85% of steel is recycled at the end of its life. Such a high figure might seem surprising until one realises that the process is enhanced by steel's natural magnetism, which makes it easy to sort.

In UK construction, the re-use and recycling rates of various steel products have been estimated at 92% for rebar, 85% for hot-dip galvanized sheet and 99% for structural steel sections. Some sections and cladding are reused in agricultural and industrial buildings especially, and this is facilitated by the use of bolted sections rather than riveting and/or welding. By saving remelting, re-use is the most environmentally advantageous approach at the end of a building's life.

The energy used in producing steel from recycled steel is roughly one-third of that for new steel. Recycling steel saves energy, CO_2 and resources by displacing the need to make more steel from virgin sources. Unfortunately though, both worldwide and in the UK, the demand for steel outstrips the supply from demolished or scrapped steel. In fact, all recovered scrap is already recycled through primary and secondary steel-making routes in one global system. As scrap is a globally traded raw material, it is impractical to distinguish for each country between primary produced steel and steel produced from scrap.

A global view is instead taken, which avoids the impracticalities of determining the precise origin of steel consumed in the UK. ISO 14041 sets the method by which the embodied energy and product life-cycle environmental impacts should be calculated. In this way, the mix of new and recycled steel and end-of-life recycling are taken into account, taking a "cradle to grave" approach to environmental consideration.

So, using UK recycled rates, the figures are 13.1 MJ/kg for steel sections and 12.1 MJ/kg for rebar. The corresponding CO_2 outputs are 0.76 kg of CO_2 per kg of steel and 0.79 kg of CO_2 per kg of reinforcement. Not included in these figures is the energy for fabrication, transport from factory to site and on-site construction, although these are relatively minor in comparison.

The argument from the concrete lobby is that although the figures reflect the worldwide situation regarding the proportion of available recycled steel versus steel from virgin resources, it plays down the impact of structural steel in the UK, which predominantly uses steel from virgin sources.

However, this new steel will in the main be reused many times, so it could be seen as unfair to account for the initial energy cost against its first life.

Most of the world's iron ore production comes from a handful of large international mining companies and many of these have systems to minimise environmental impact.

Reinforced concrete

The embodied energy of producing concrete is about 380 kg of CO2 per m^3 concrete in structural components such as floors and columns. It is about 310 kg of CO_2 per m^3 concrete in pad foundations or the like. Increasingly, though, cement may be partly replaced by alternatives such as pulverised fuel ash (PFA), a by-product of coal-fired power stations, and ground granulated blast furnace slag (GGBS), a by-product of steel production.

A substitution of cement with 30% PFA saves about 20% CO_2, whereas substitution with 50% GGBS saves about 40% of CO2, but this assumes that CO_2 should be entirely accounted for in steel manufacturing figures, rather than the GGBS that flows from it.

The Concrete Centre says:

- 85% of aggregate travels less than 30 miles
- 90% of cement is sourced from the UK, whereas 10% is imported
- In the UK, almost all reinforcement is produced from recycled steel
- All the companies that produce cement have environmental management systems in place and programmes to minimise the environmental impact from mining activities.

It is estimated by BRE's Green Guide that 50% of concrete is crushed and recycled, 40% is downcycled for use such as hardcore in substructure works or road construction and the remaining 10% is waste that goes to landfill. Down-cycling does help to reduce the use of aggregates, but does not help reduce the supply of materials for

new concrete.

Ecopoints

An article like this cannot analyse all environmental impacts. For example, there are many other types of gas emissions that should be considered, such as nitrous oxide (NO_4). The best overview of the overall impact of these materials is the Ecopoint rating developed by BRE.

- Structural steel has 11 Ecopoints per tonne
- Reinforced concrete to 35 N/mm^2 (including rebar at 100 kg/m^3) has 5.3 Ecopoints/m^3 (using a density of 2371 kg/m^3), or 12.57 Ecopoints per tonne.

Environmental summary

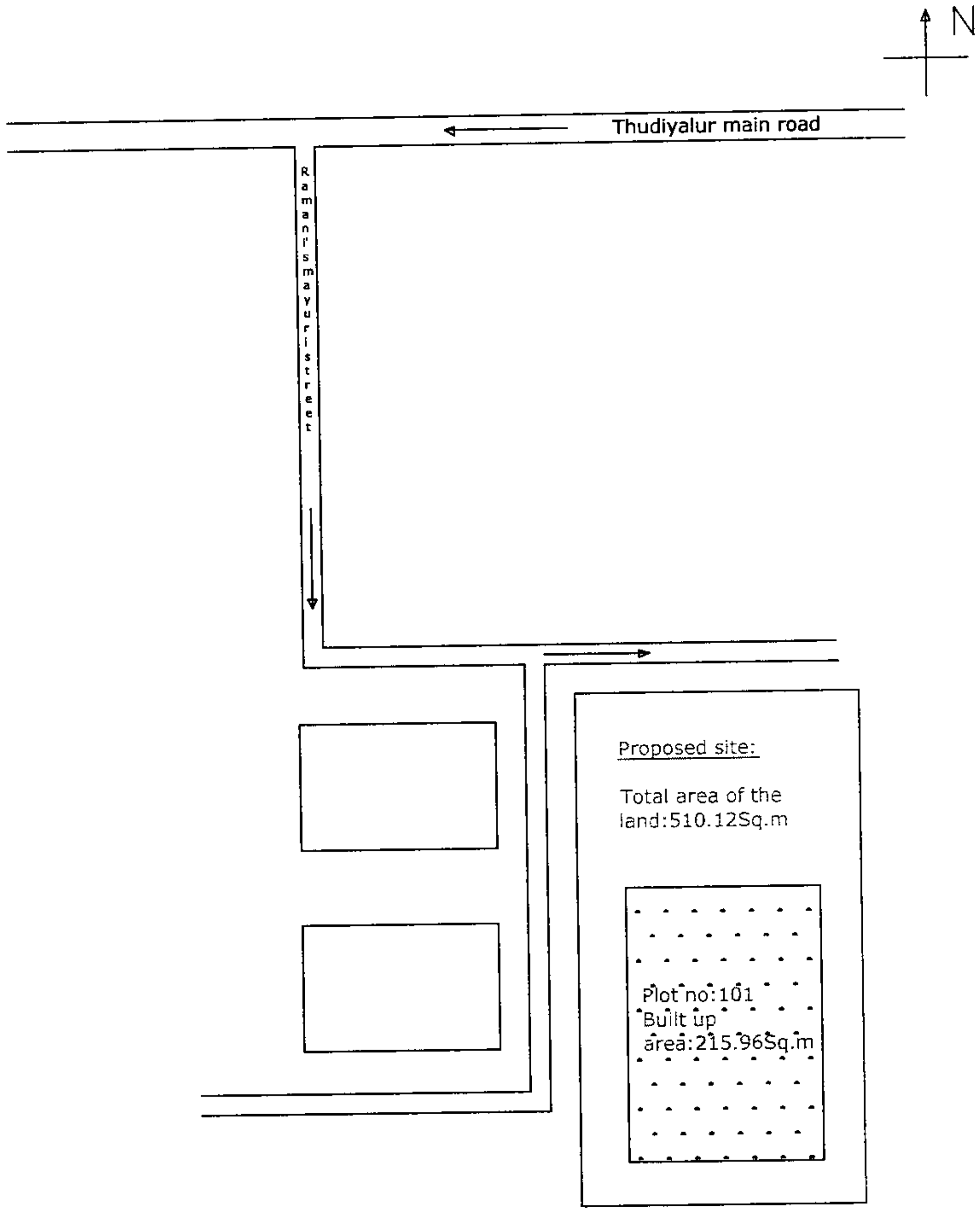
If we ignore operational energy savings, the concrete option appears to be about 30% worse (see table overleaf), but when operational energy is accounted for, this dwarfs the embodied energy and the appraisal is reversed showing a saving of 6% for the concrete option.

2. CONCLUSION

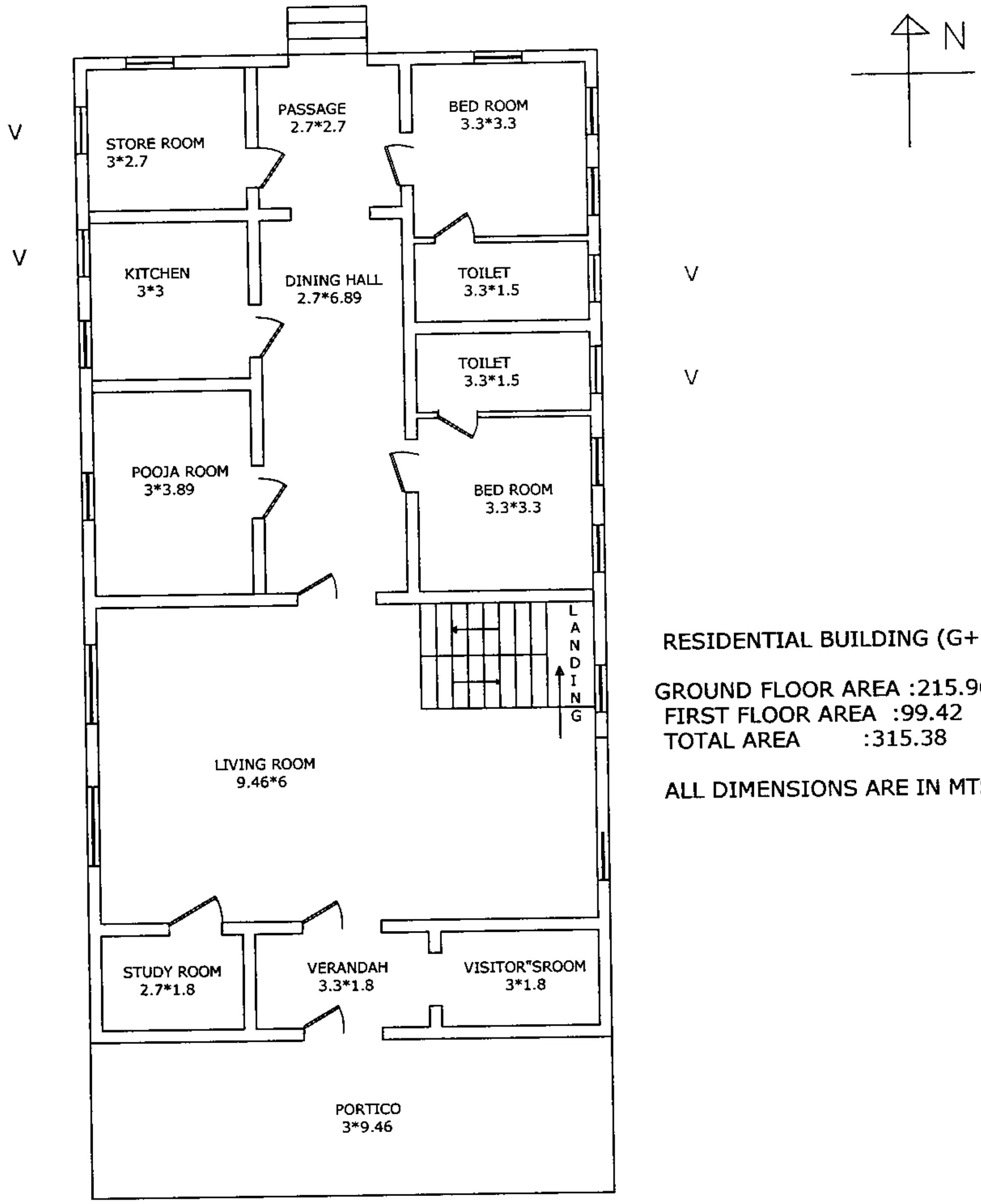
Constructional steel presents a lot of positive environmental impacts regarding its sustainability, refurbishment and reusability issues. It also presents a lot of advantages in making energy efficient buildings. Additionally, when considering the earthquake resistant, durable and easily reusable or dismantling buildings; steel construction becomes a very strong building alternative. It gains a special importance concerning the required structural performance, damaged building reinforcement and waste material management especially in the earthquake areas. So which option is best, steel or concrete? Given the variety of factors to consider, there's not a standard answer to this ongoing debate. Both steel and concrete form safe and durable buildings, but timing and market conditions could dictate that one alternative makes more sense for a specific project. With the many pros and cons for each alternative, owners are wise to give careful consideration to all factors before making an informed decision.

REFERENCES

- Design Aids for Reinforced Cement concrete IS 456:1978.
- Indian Standard Code of Practice for plain and Reinforced Cement concrete IS 456:2000.
- Indian Standard Code of Practice for loads;
    - IS 875 – 1987 (PART- I)
    - IS 875 – 1987 (PART- II)
- V.L.SHAH and S.R.KARVE a book on "Illustrated Design of Reinforced Concrete Buildings"
- VARGHESE, a book on "Limit State Design of Reinforced Concrete elements".
- Dr. RAM CHANDRA and VIRENDRA GEHLOT, a book on "Design of Steel Structures".
- VAZRANI, a book on "Steel Structures".
- National Building Code.
- STAAD PRO manual for practice.


N
Thudiyalur main road
Ramani's mayuri street
Proposed site:
Total area of the land:510.12Sq.m
Plot no:101
Built up area:215.96Sq.m

SITE PLAN

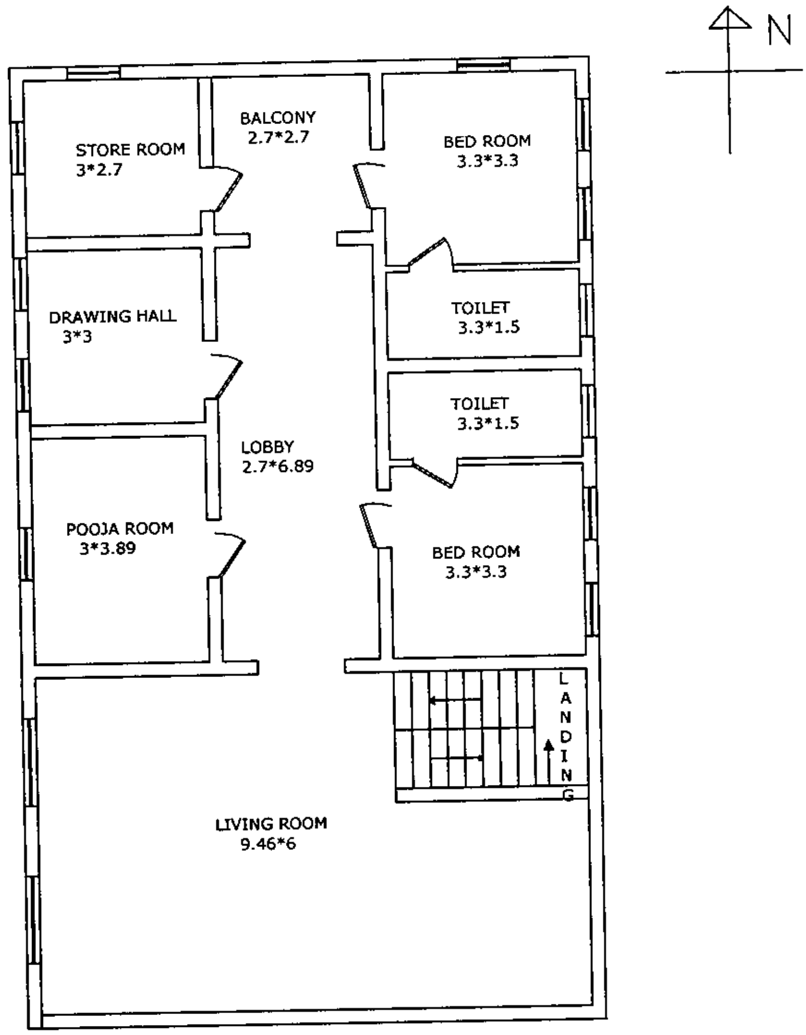


RESIDENTIAL BUILDING (G+1)

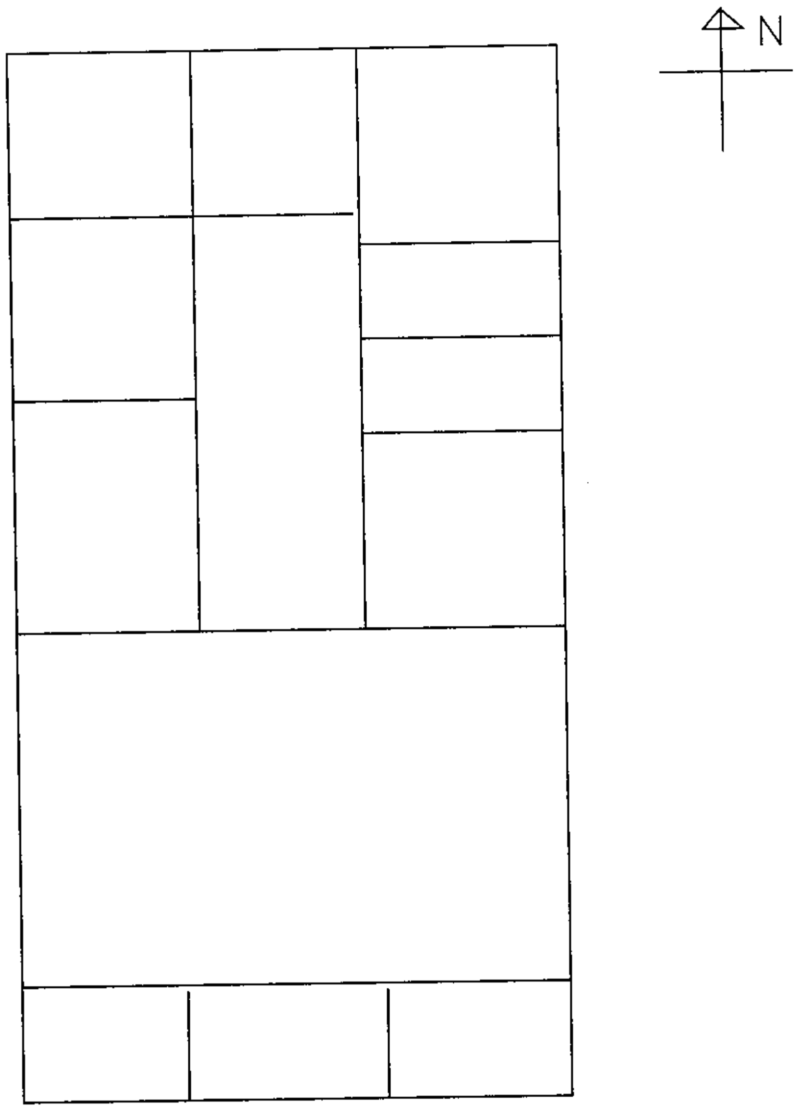
GROUND FLOOR AREA :215.96
FIRST FLOOR AREA :99.42
TOTAL AREA :315.38

ALL DIMENSIONS ARE IN MTS

GROUND FLOOR

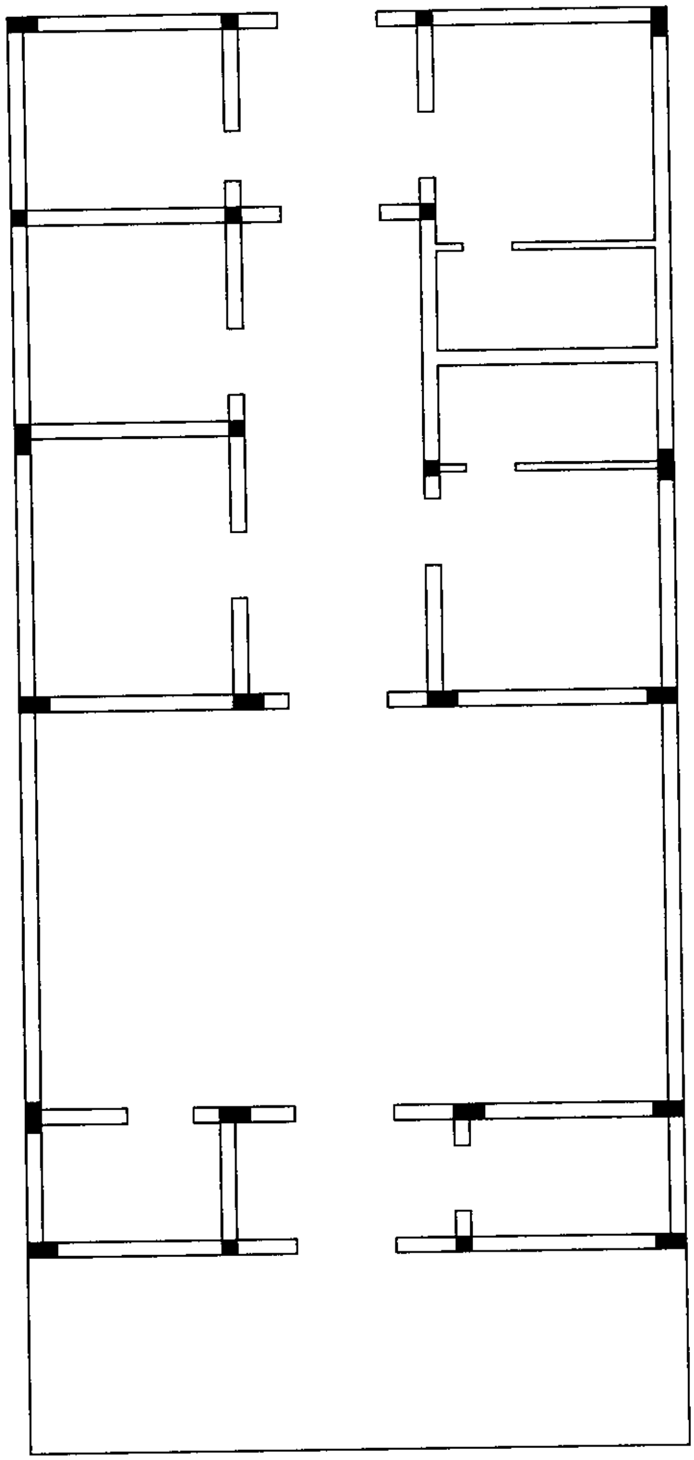

N
STORE ROOM
3*2.7
BALCONY
2.7*2.7
BED ROOM
3.3*3.3
DRAWING HALL
3*3
TOILET
3.3*1.5
TOILET
3.3*1.5
LOBBY
2.7*6.89
POOJA ROOM
3*3.89
BED ROOM
3.3*3.3
LANDING
LIVING ROOM
9.46*6

FIRST FLOOR



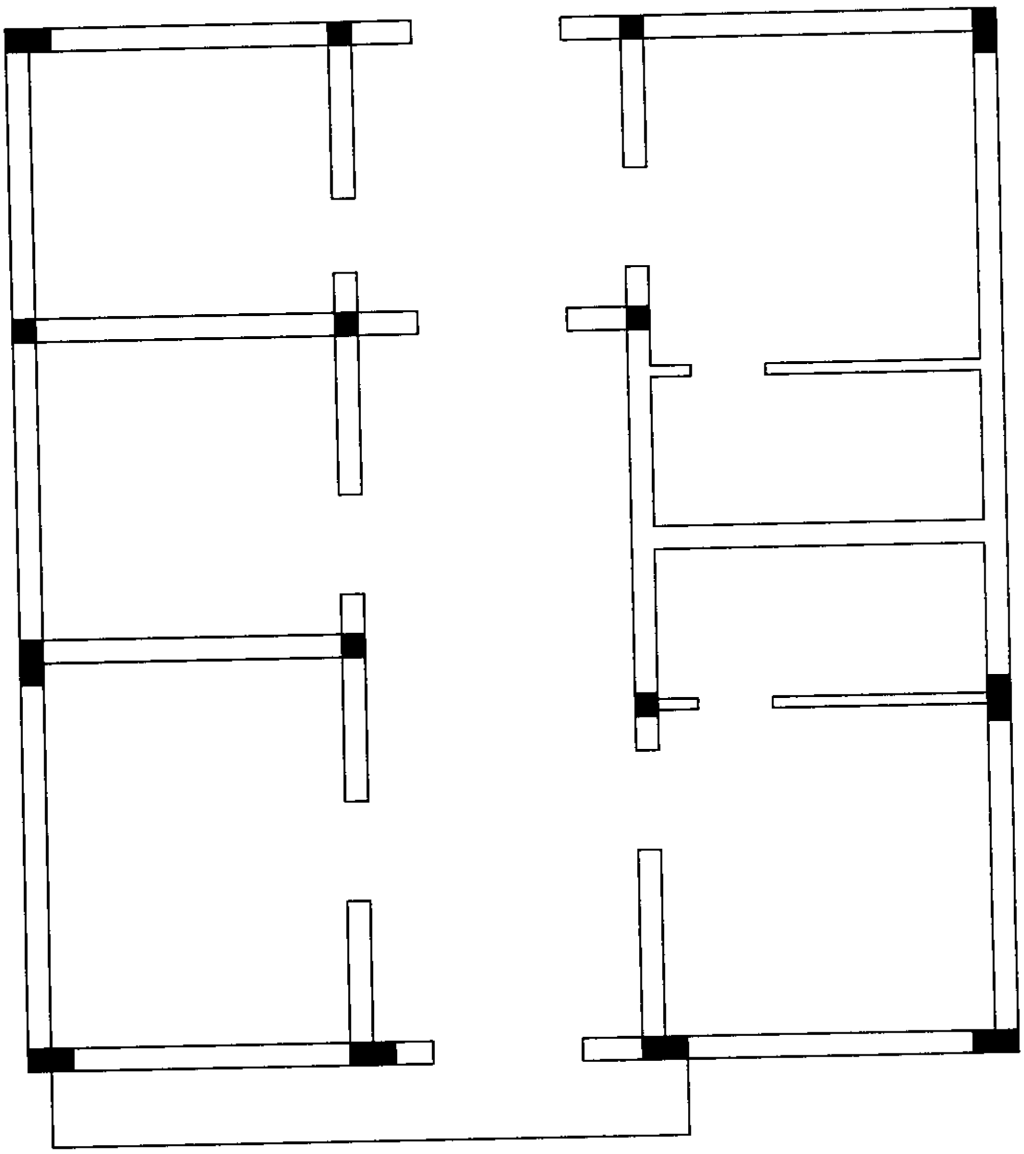
CENTRE LINE DIAGRAM

COLUMN ORIENTATION



GROUND FLOOR

COLUMN ORIENTATION



FIRST FLOOR